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Underwater Storage of Combined Sewer Overflows

172

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Two 100,000 gallon collapsible tanks were anchored under water in Lake Erie to demonstrate, on a pilot scale, the feasibility of storing combined sewer storm overflow in underwater flexible tanks. Site selection, model testing, system design, construction, and one year's operation were included in the investigation. During the year's operation, 988,000 gallons of storm overflow were contained and returned to the treatment plant for treatment. Preliminary construction costs of $1.88 per gallon of storage capacity showed promise of decreasing to $0.40 per gallon for optimum system sizes. Operation and maintenance costs must be kept minimal, with automatic operation, and only essential utilization of electrical power. It was determined that for larger installations, modular systems of 250,000 to 500,000 gallon basic unit tanks should be employed, with site characteristics determining both the overall feasibility of the system, and the optimum tank size should such a system be feasible. Life expectancy for tank materials can only be determined in full scale operation in the actual environment. The Sandusky Pilot Facility contained over 90% of the pollutional load which ordinarily would have been discharged and delivered it for treatment at the treatment plant. (Lowry-Texas)

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*Water pollution control, *Storm runoff, *Storage tanks, Pilot plants, Construction costs, Operation and maintenance, Flexibility, Optimization, Performance, Waste water Treatment, Ohio,

17b. Identifiers

*Combined sewer overflows, *Fabric

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UNDERWATER STORAGE
OF
COMBINED SEWER OVERFLOWS

by

Karl R. Rohrer Associates, Inc.
529 Grant Street
Akron, Ohio 44311

for

ENVIRONMENTAL PROTECTION AGENCY

Program # 11022 ECV
Contract # 14-12-143

September 1971

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This report has been reviewed by the Environmental Protection Agency and approved for publication. Approval does not signify that the contents necessarily reflect the views and policies of the Environmental Protection Agency, nor does mention of trade names or commercial products constitute endorsement or recommendations for use.

ABSTRACT

The purpose of this study was to demonstrate off-shore underwater temporary storage of storm overflow from a combined sewer in flexible tanks. Site selection, model testing, system design, construction, and one year's operation were conducted under the study.

A pilot demonstration facility was constructed in Sandusky, Ohio where combined sewer overflow from a 14.86-acre residential drainage area was directed to two-100,000 gallon collapsible tanks anchored underwater in Lake Erie. The stored overflows were pumped back to the sewer system after a storm event for subsequent treatment. During the year's operation, a total of 988,000 gallons of storm overflow was contained and returned for treatment.

As constructed, the facility cost was about $1.88 per gallon of storage capacity while future projections indicate costs of less than $0.40 per gallon possible.

Evaluation of the underwater storage system in controlling combined sewer pollution, comparison of cost with other storage methods and other combined sewer pollution control methods, operational difficulties and recommendations of an improved system are included in the study report.

This report was submitted in fulfillment of Contracts 14-12-25 and 14-12-143 between Water Quality Research, Environmental Protection Agency, and Karl R. Rohrer Associates, Inc.

Pursuant to Executive Reorganization Plan Number Three of 1970, effective December 2, 1970, and Environmental Protection Agency Order Numbers 1110.1 and 1110.2, all references to Federal Water Quality Administration or Federal Water Pollution Control Administration herein shall be to the Environmental Protection Agency, Water Quality Research.

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SECTION I

CONCLUSIONS

1. Off-shore temporary underwater storage of storm overflow from combined sewers in flexible underwater storage tanks is feasible and was demonstrated successfully in the pilot facility at Sandusky, Ohio.

2. In order to minimize operation and maintenance costs, the system must be gravity fed, a minimum of pretreatment prior to the storage tank must be possible, electricity cost for pumps, compressors, and instrumentation must be kept to a minimum, and the system must be able to operate automatically with a minimum of scheduled maintenance.

3. Underwater storage of combined sewer overflows can be competitive with other methods of storage where land is not available and physical and hydraulic site characteristics permit. River and lake installations have different controlling design parameters.

4. A modular tank system should be designed for larger drainage areas. For economical use of the underwater storage system, a 250,000 gallon to 500,000 gallon basic unit should be used. Site characteristics would determine optimum tank size.

5. Life expectancy for neoprene rubber coated nylon fabric (or other) must be determined in full scale operation in the environment imposed. Aging and subsequent loss of fabric strength must be determined.

6. Insufficient data is available on pollution loads from combined sewer overflows and pollution loads from storm sewer overflow.

SECTION II

RECOMMENDATIONS

General

1. Further operation of the pilot facility for a minimum of one year should be completed to acquire sufficient data for pollution control evaluation.

2. Study of detention time of stored liquid should be completed to determine action of underwater storage on overflows.

3. Investigation of further combined sewer overflows should be studied in detail for application of off-shore temporary underwater storage of combined sewer overflows in a full scale application and to determine combined sewer pollution loads. Separate storm sewer pollution loads should also be determined for a similar area.

4. Collection of grab samples of all flows should be used liberally to confirm results from automatic samplers.

System Design

1. The fabric elongation in the two-ply neoprene coated nylon fabric should be limited to six percent working elongation. Elongation up to twelve percent should be studied. Physical and chemical properties of the fabric on aging in the proposed environment should be studied.

2. Tank design should be modular with a 250,000 gallon basic design capacity. Where site conditions permit higher tank capacities up to about 500,000 gallons might be used.

3. On future installations only the following items would be required:

   a. Site preparation,
   b. Connection chamber with bar screen,safety overflow, and piping to underwater tank,
   c. Tank and anchorage,
   d. Pressure relief valve,
   e. Underground pump station,

Preceding page blank

    f. Tank full indicator to close influent control valve.

4. Check valves should, where necessary, be included on the influent pipe beyond the influent control valves.

5. Future installations could use an underground prebuilt pump station for pumps and control structure.

6. The flushing system to help remove sediment from within the tank could possibly be eliminated in future designs.

7. The tank level control system should be replaced by a non or minimum power using method. It is not necessary to continuously monitor tank volume. Control valve operation could be initiated by internal pressure or fabric tension sensors.

8. Only one gas vent valve need be used with existing tank shape and design. The gas vent valve could use soft seats and a plastic ball float.

9. The pressure relief valve should be used on all storage tanks. An adaptation of a commercially available magnetic-gravity valve might be more maintenance free.

10. Future underwater storage systems should have ability to fill the tank with bay or river water and should be checked annually. Divers should check each installation periodically.

SECTION III

INTRODUCTION

Purpose of Project

The past decade has brought an increasing awareness of the pollution load contributed to the nation's water resources by combined sewer collection systems. Many cities adopted the combined manner of sewer construction due to an economic savings when compared to separate storm and sanitary sewers. As a result, over 54 million people in 1329 jurisdictions with a total area of 3,029,000 acres are served by combined sewer systems in the United States today.

A basic point to remember is that combined sewers are/have been designed to overflow. Under today's condition of increasing urban growth, a spreading asphalt and concrete barrier increases runoff volumes and rates and these overflows increase in frequency and duration.

Methods are being studied on how to eliminate the pollution problem presented by combined sewer overflows. Separation of sewers is economically unfeasible and does not allow treatment of the storm runoff. Complete treatment at each outfall is not feasible.

This project demonstrates a method of reduction of combined sewer pollution. Since most regulator - outfalls are at a river or lake, temporary storage of the storm overflow in underwater flexible tanks with pumping of the overflow to the existing interceptors and to existing water pollution control stations for treatment during non-peak hours is a possible method of control.

Underwater storage, where applicable, offers lower land costs, a more aesthetically pleasing and compatible installation, and an economical method of elimination of combined sewer pollution.

Scope of Project

The Scope of the Project was broken down into three distinct areas. The first area of work consisted of:

1. Site investigations,
2. Aquisition of property use and access rights,

3. Topographic and hydrographic surveys,
4. Hydraulic studies,
5. Hydraulic, structural, and instrumentation design, and,
6. Specifications and construction drawings.

The second area of work consisted of:

1. Construction of pilot facility.

The third area of work consisted of:

1. Operation of pilot facility for one year,
2. Evaluation of one year of operation, and
3. Removal or continued operation of pilot facility.

Project Objectives

The primary objectives of the project were to:

1. Obtain an operable system and operating procedures,
2. Find the total cost of storage and the cost/pound of BOD reduction and compare underwater storage with other methods of storage and treatment including sewer separation, and
3. Determine the degree of pollution control provided.

SECTION IV

SITE INVESTIGATION

The first phase of the work to be completed under the demonstration pilot facility study was the selection of a site where the facility could be constructed, the acquisition of property use and access rights, topographic and hydrographic surveys, hydrologic studies, hydraulic, structural and instrumentation design, and preparation of complete specifications and construction drawings.

Site Selection

The F.W.P.C.A. limitations imposed on site selection were:

1. Thirty acre combined sewer drainage area,
2. Simple non-mechanical regulator device,
3. Suitable water body conditions,
4. Minimum interference with waterway uses,
5. Ample workspace and access,
6. Property and right-of-way rights and permits available,
7. Adjacent area compatible and installation acceptable to public, and
8. Total storage capacity 200,000 gallons.

Table 1 lists nine possible site locations presented to the F.W.P.C.A. during January, 1967. The locations of these sites are shown in Figure 1.

The nine potential site locations were reduced to the best four locations and these were presented to the F.W.P.C.A. during February, 1967. Table 2 describes these sites. Rohrer recommended that the Sandusky, Ohio site be approved as the demonstration pilot facility site from the preliminary investigations conducted.

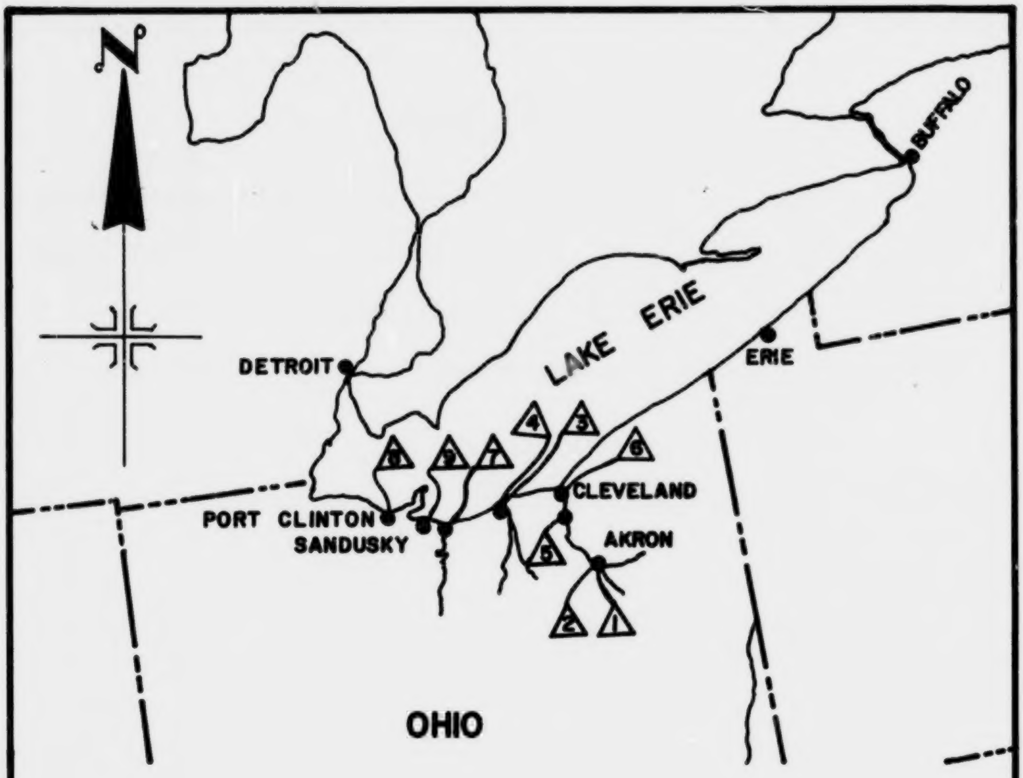
Tentative approval of the McEwen Street combined sewer drainage in Sandusky, Ohio area was received from the F.W. P.C.A. and work was begun on obtaining all permits and permissions necessary for the project. Design of the system could not be completed until after site selection had been accomplished. However, preliminary design plans had to be completed to acquire permissions and permits necessary.

TABLE 1
SITE INVESTIGATION
Sites Proposed

1. Akron, Ohio . . . . Riverside Blvd. Sewer District
2. Akron, Ohio . . . . Spalding-Weaver-Evers & Tallmadge Sewer District
3. Avon Lake, Ohio . . Southpoint Sewer District
4. Avon Lake, Ohio . . Moorewood Avenue Outfall Sewer
5. Brecksville, Ohio
6. Cleveland, Ohio . . W. 110th Street Outfall Sewer
7. Huron, Ohio . . . . Outfall Sewer at Treatment Plant
8. Port Clinton, Ohio. Outfall Sewer at Water Works
9. Sandusky, Ohio. . . McEwen Street Outfall Sewer

Site No.	1	2	3	4	5	6	7	8	9
Site Criteria									
A. Strictly Combined Sewer	X	X	X	X	O	X	O	X	X
Estimated Drainage Area	28	95	85	290	45	29	65	69	22
(Acres)	X	O	O	O	O	X	O	O	X
B. Water Body Depth	X	O	X	X	O	X	X	X	O
Flood Velocity	X	O	X	X	O	X	X	X	X
Bed Material	O	X	X	X	X	O	X	X	X
C. Interference with Other Water Way Uses	X	X	X	X	X	X	O	O	X
D. Ample Work Space	X	X	X	X	X	O	O	X	X
Convenient Access	O	X	X	X	O	O	O	X	X
E. Permissions	X	X	O	X	X	O	X	X	X
F. Compatible Adjacent Area	X	X	X	X	X	O	O	O	X

Key
X - Indicates Good Condition
O - Indicates Unfavorable Condition


N
DETROIT
LAKE ERIE
BUFFALO
ERIE
PORT CLINTON
SANDUSKY
CLEVELAND
AKRON
OHIO
8
9
7
4
3
6
5
2
1

PRELIMINARY SITE LOCATIONS

KARL R. ROHRER ASSOCIATES, INC.
AKRON, OHIO

FIGURE 1

TABLE 2
FINAL SITE SELECTIONS
Sites Proposed

1. Akron, Ohio . . . . Riverside Blvd. Sewer District

2. Cleveland, Ohio . . . . . W. 110th Street Outfall Sewer

3. Port Clinton, Ohio. . . . Outfall Sewer at Water Works

4. Sandusky, Ohio. . . . . . McEwen Street Outfall Sewer

Site No.	1	2	3	4
Site Criteria				
A. Strictly Combined Sewer	X	X	X	X
Estimated Drainage Area (Acres)	28	29	69	22*
	X	X	O	X
B. Water Body Depth	X	X	X	O
Flood Velocity	X	X	X	X
Bed Material	O	O	X	X**
C. Interference with other water way uses	X	X	O	X
D. Ample Work Space	X	O	X	X
Convenient Access	O	O	X	X
E. Permissions	X	O	X	X
F. Compatible Adjacent Area	X	O	O	X

Key

X - Indicates Good Condition
O - Indicates Unfavorable Conditions

* - With final study of the Sandusky site area, it was found that the drainage area contains 14.86 acres.

**- With soil borings taken, unfavorable bedrock conditions existed.

Project Approvals

Permission or permits were required from the companies and agencies listed in Table 3. All stated no objection to the installation.

When continued operation after the one-year operation period was considered, it was necessary to reobtain several permits for the project since the original request for approvals for the installation had been based on a one-year operation period with subsequent system removal. The major approval reobtained was from the U.S. Army, Buffalo District, Corps of Engineers since their permit was only good until December 13, 1969. On or prior to that date, the installation had to be removed. An extension of time for operation of the installation was requested; and, on January 7, 1970, a new permit was issued requiring removing of the installation on or prior to December, 1972.

Lake Erie

Lake Erie is the most southerly as well as the shallowest of the five Great Lakes. Lake Erie covers a total of 9,910 square miles and has a drainage basin consisting of 32,630 square miles. The distance from Buffalo, New York at the easterly end of the lake to Toledo, Ohio at the westerly end of the lake is 241 miles with the greatest width being about 57 miles.

The maximum depth in Lake Erie is 210 feet at a point just southeast of Long Point, Ontario and it has an average depth of 58 feet over the entire lake. Generally, the deepest part of the lake is at the eastern end, while the island region at the westerly end is the most shallow.

During the winter, very heavy ice forms along the shore line and extends some distance into the lake. The westerly end, or island region, is often quite solidly iced over.

The water temperature of Lake Erie fluctuates from about 75° in the late summer or early fall to 32° during winter and early spring.

The maximum wind velocity recorded on Lake Erie was 74 knots (85 MPH) on June 10, 1963. However, recorded wind velocity data was only begun in 1941; and, over a longer data interval, this maximum wind velocity would with certainty increase.

Lake Erie Level

The average or normal elevation of the surface of Lake Erie

TABLE 3
PROJECT APPROVALS

Agency	Interest
1. Department of the Army Buffalo District Corps of Engineers Buffalo, New York	Jurisdiction over all construction in navigable waters in Lake Erie.
2. Ninth Coast Guard District Cleveland, Ohio	All structures in navigable waters must be marked in accordance with law.
3. City of Sandusky Sandusky, Ohio	Permission to connect to existing combined sewer outfall. Coordination with City of Sandusky Water Pollution Control Station.
4. Farrell-Cheek Steel Co. Sandusky, Ohio	Upland property owner to East of site.
5. Penn Central Cleveland, Ohio	Upland property owner to South of site. Owner of leased property for facility.
6. Ohio Department of Health Columbus, Ohio	Approves all plans for water pollution control stations and associated facilities in the State of Ohio.
7. Regional Director Bureau of Fish and Wildlife U.S. Department of Interior	Protection of fish and wildlife.
8. Ohio Department of Natural Resources Division of Water	Jurisdiction over all natural water bodies in State of Ohio.
9. Ohio Department of Natural Resources Division of Watercraft	Jurisdiction over all watercraft in Ohio Waters. Interest in marking of tank location.
10. Ohio Departments of Public Works Columbus, Ohio	Land in Lake Erie is under jurisdiction of this agency.

varies irregularly from year to year. During the course of each year the surface is subject to a consistent seasonal rise and fall, the lowest stages prevailing during the winter months and the highest stages during the summer months.

In the 110 years from 1860 to 1969, the difference between the highest (572.76) and the lowest (567.49) monthly mean stages of the whole period has been 5.27 feet; the greatest annual fluctuation as shown by the highest and the lowest monthly means of any year was 2.75 feet; and the least annual fluctuation was 0.87 foot. (International Great Lakes Datum, 1955. Elevations are in feet above mean water level in Gulf of St. Lawrence at Father Point, Quebec. All elevations in this report are referred to this datum.)

In addition to the annual fluctuations there are also oscillations of irregular amount and duration produced by storms. Some with periods of a few minutes to a few hours are the results of squall conditions. The fluctuations are produced by a combination of wind and barometric pressure changes that accompany the squalls. At other times, the lake level is affected for somewhat longer periods, such as many hours or days by strong winds of sustained speed and direction which drive the surface water forward to raise its level on the lee shore and lower it on the weather shore. This type of fluctuation has a very pronounced effect on Lake Erie, because it is the shallowest of the Great Lakes and afford the least opportunity for the impelled upper water to return through reverse currents beneath the depth disturbed by storms. As a result, the water level in the harbors, particularly those near each end of the lake fluctuates markedly under the influence of winds, varying with the direction, strength, and persistence. The maximum effect occurs at Sandusky and Toledo, Ohio and at the mouth of the Detroit River.

Sandusky, Ohio

The City of Sandusky, Ohio is situated on the southeastern shore of Sandusky Bay near the western end of Lake Erie. The City of Sandusky is located approximately midway between Cleveland, Ohio and Toledo, Ohio at a latitude of 41°27' north and a longitude of 82°43' west.

Sandusky presently incorporates 9.1 square miles of land area and 5.8 square miles of water area. The elevations of the city vary between 575 and 603 feet. The land area is extremely flat. Few slopes exist which are greater than 1%. The estimated 1969 population was 36,479.

Sandusky Bay provides a natural harbor for shipping.

The City of Sandusky has over 11 miles of waterfront; and, in 1968, the port ranked second on Lake Erie in coal tonnage shipped.

The City of Sandusky is a center for water oriented recreation. Adjacent to the City of Sandusky, are East Harbor State Park and Cedar Point which offer recreation to residents of Northern Ohio and many surrounding states. Both provide beaches for swimming which are becoming scarce in Lake Erie.

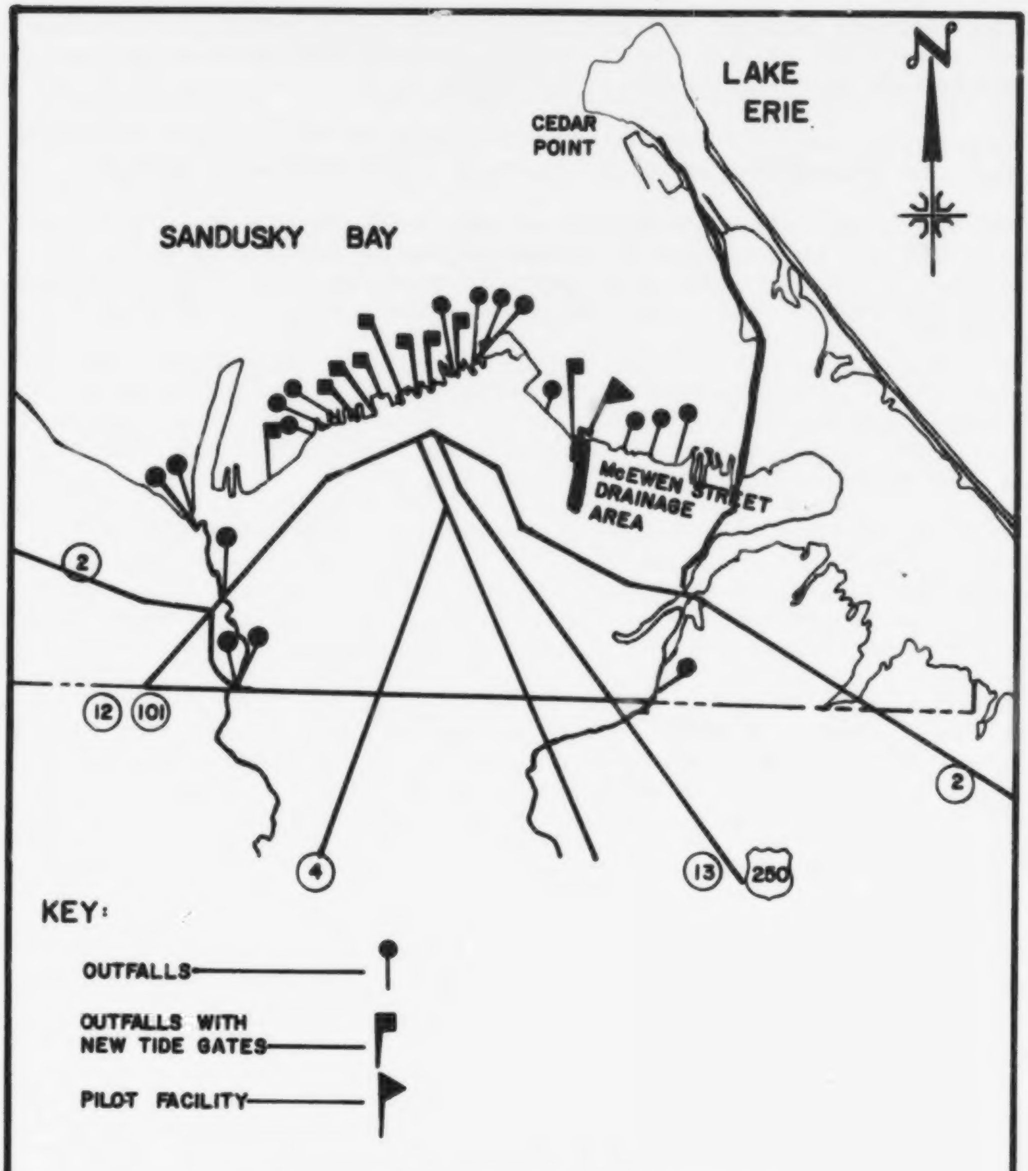
Water Pollution Control Station and Sewage Collection Sewage

At the present time, the City of Sandusky Water Pollution Control Station offers primary treatment only. However, construction of secondary treatment facilities with phosphate removal is underway.

The City of Sandusky has developed the future plans for the Water Pollution Control Station on a regional basis so that both City and Erie County sanitary wastes will be treated here. Erie County will provide proportional funds for secondary treatment and future expansion requirements. A tributary area of 5,863 acres of City land area and 22,638 acres of County land area are presently anticipated to be served by the City of Sandusky Water Pollution Control Station. A design population of 83,000 for the year 2000 has been used as the basis for secondary treatment design.

The 1967 average daily flow at the Water Pollution Control Station was 6.2 MGD: the 1968 average daily flow was 7.7 MGD. The average daily flow for the first 9 months of 1969 was 11.9 MGD. Of these average flows, only about 4.5 MGD is sanitary sewage. The remainder is estimated to be storm water and backwater from Sandusky Bay.

The baywater from Sandusky Bay is a problem which was tackled by the city in 1969. A total of 25 outfalls exist in the combined sewer collection system. Some of the storm overflow structures incorporating leaping weir devices have elevation below the abnormally high bay levels. In the summer of 1969, nine of the worst outfalls had new headwalls constructed and tide gates installed to prevent this backflow. Eight of these installations were in operation by July 31, 1969. Average daily flow at the Water Pollution Control Station dropped from 20.8 MGD for June to 10.0 MGD for July and 7.6 MGD for August. Figure 2 shows the location of the 25 outfalls and the location of the new tide gates.


N
LAKE
ERIE
CEDAR
POINT
SANDUSKY BAY
McEWEN STREET
DRAINAGE
AREA
2
12
101
4
13
250
2
KEY:
OUTFALLS
OUTFALLS WITH
NEW TIDE GATES
PILOT FACILITY

COMBINED SEWER OUTFALLS
SANDUSKY, OHIO

KARL R. ROHRER ASSOCIATES, INC.
AKRON, OHIO

FIGURE 2

The present sewage collection system contained 110.74 miles of sewers in 1968. About 53.6 miles of the system is comprised of combined sewers.

Presently, any additional areas added to the collection system are sewered with separate storm and sanitary sewers.

However, the older sections of the city have combined sewers. A total of 24 individual combined sewer districts with 25 outfalls exist. Combined sewer districts serve approximately 2,205 acres of the total of 5,863 acres of City land area.

Much has been said about the pollution of Lake Erie. Sandusky Bay has felt the effects of the municipal sewage treatment plant effluent which flows to the Sandusky River and to the Sandusky Bay. Sludge deposits have built up which are polluted with a high organic content.

With municipalities going to secondary treatment along the Sandusky River and Sandusky Bay, reduction of the sludge deposits and subsequent pollution should result. In August, 1968, the report from the F.W.P.C.A., Great Lakes Region Office, "Lake Erie Report-A Plan for Water Pollution Control," states that Sandusky Bay is enriched along the waterfront of the City of Sandusky and the eastern portion of the Bay is polluted by septic sludge, algal growth, scum, and bacteria. The combined sewer overflows are considered to contribute a large portion of the pollution load. It was estimated in this report that the pollution load from the combined sewers in this area will double by the year 2020 if no further control measures are enforced.

In 1967, 600,000 cubic yards of dredgings were removed from Sandusky Bay to keep the harbor open. The controversial dumping of this material in Lake Erie has been condemned by the F.W.P.C.A., Great Lakes Region Office, due to the pollution load exerted on Lake Erie.

The Ohio Water Pollution Control Board has set water quality standards for Lake Erie which include Sandusky Bay and for the Sandusky River. These water quality standards are included in the Appendix, page 145. Further increases in water quality standards are expected to be enacted in the future. Hearings were conducted on February 25, 1970 by the Ohio Water Pollution Control Board on upgrading general water quality standards now in effect in the areas of pH, D.O., temperature, and coliform count, which would upgrade the Sandusky River Standards.

Sandusky Bay

Sandusky Bay is located on the southwestern shore of Lake Erie. The bya entrance is approximately 50 miles westerly from the Cleveland harbor entrance. The bay entrance is formed by and the Bay is protected by Cedar Point on the east and Point Marblehead on the west. Figure 3 shows the Bay area.

Sandusky Bay has a surface area of about 22.5 square miles. The Bay is about 16 miles long from the mouth of the Sandusky River to the Bay entrance. The Bay is broken into two sections by State Route 2 which crosses at the narrowest point. The Bay width varies from about 5.6 miles to 1.2 miles.

Sandusky Harbor is kept open by dredging the channel from Lake Erie through the Bay to the waterfront at Sandusky, Ohio. The harbor channel and turning basin have been originally excavated through bedrock.

The water depth in Sandusky Bay when referenced to the low water datum, 568.6 feet, varies from about nine to two feet over most of the bay except in the shipping channel and turning basin.

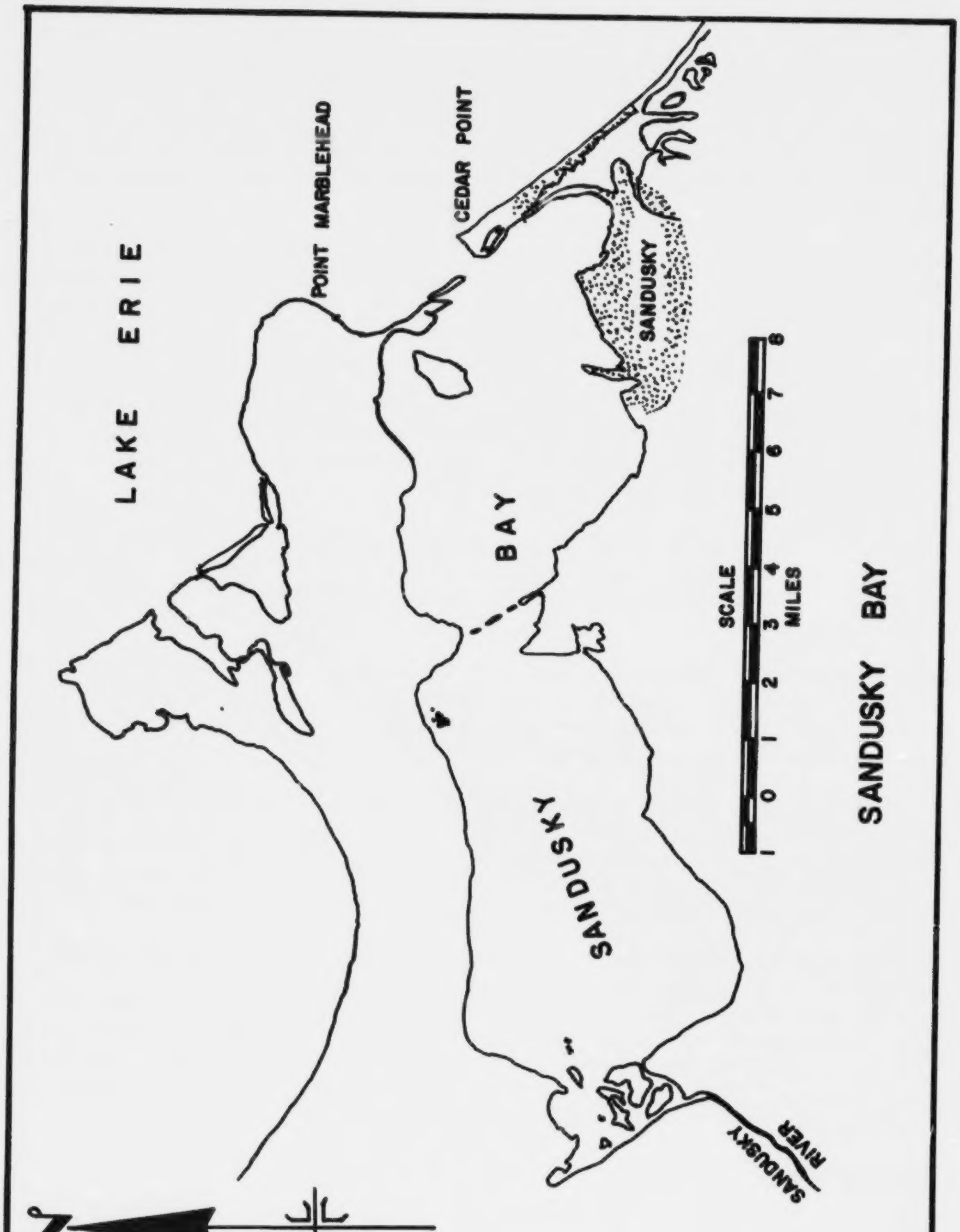
Water currents exist in the bay during periods of rising and falling waters. However, at the site of the underwater storage system these currents are negligible. Littoral dirft due to wave action does occur.

Sandusky Bay freezes over each winter. In preliminary investigations, it was determined that the ice along the underwater storage tank area does not normally pile up along the shore as it does along other sections of Lake Erie shoreline. No piling up of ice occurred in 1967, 1968, or 1969.

The physical shape of Sandusky Bay yields fetch distances of about 3.2 miles to 1.2 miles for wind generation of waves for winds from Northwest to Northeast for design of the underwater storage tanks. However, the shallowness of the bay limits the size of waves which can be generated by the winds. Water depth in the bay limits the design wave choice for the underwater tank design.

Sandusky Bay Level

Data studied showed that strong winds produce abnormal water level fluctuations in Sandusky Bay. This fluctuation is due to the shallow bay conditions. A high water elevation of 574.1 in 1952 and a low water elevation of 566.9


LAKE ERIE
POINT MARBLEHEAD
CEDAR POINT
SANDUSKY
SANDUSKY BAY
SANDUSKY
SCALE
MILES
1 0 1 2 3 4 5 6 7 8
SANDUSKY RIVER
SANDUSKY BAY
N

KARL R. ROHRER ASSOCIATES, INC.
AKRON, OHIO

FIGURE 3

in 1942 were recorded at Battery Park. Cleveland records are 572.74 in 1952 and 567.5 in 1936 for the same record levels.

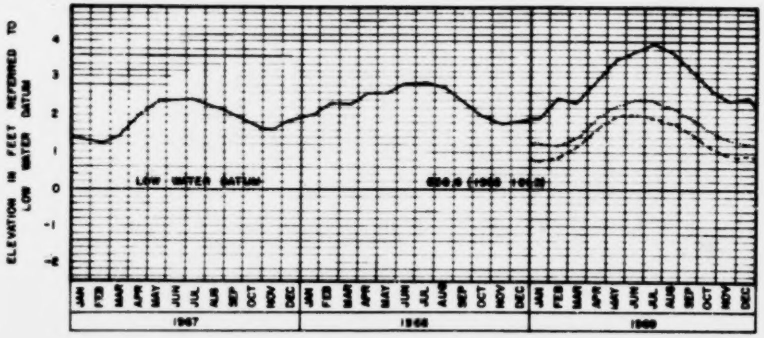
The City of Sandusky takes daily readings of the bay level at Battery Park. This data is available from 1940 to present date and was secured to provide the required information for design. The City of Sandusky has determined a statistical curve for the Lake Erie water level based on mean monthly level for 1860 to 1959. This statistical curve is presented in Figure 4-A. However, comparison of Marblehead and Cleveland, Ohio daily records for the level of Lake Erie with Battery Park records for Sandusky Bay show that while the water levels agree as to the trends, the actual water levels may vary by two feet.

Preliminary studies for the underwater storage system began early in 1967. Final design was completed during 1968 and construction began in July of 1968. Figures 4-B and 4-C show Lake Erie projected water levels for the next six months for January 1967; June, 1967; December, 1969, and the actual levels from January, 1967 to January, 1970 respectively. Projected levels of the Great Lakes are published by the Department of the Army, Lake Survey District, Corps of Engineers' "Monthly Bulletin of Lake Levels." Montyly lake levels are shown along with projected probable lake levels for the next six month period.

The high lake levels encountered in 1969 are even more unusual when the record low water levels which occurred in 1965 are considered. In only five years, the level of Lake Erie went from record low levels to record high levels. The normal high-low water level cycle is usually twice this length.

Climatic Conditions

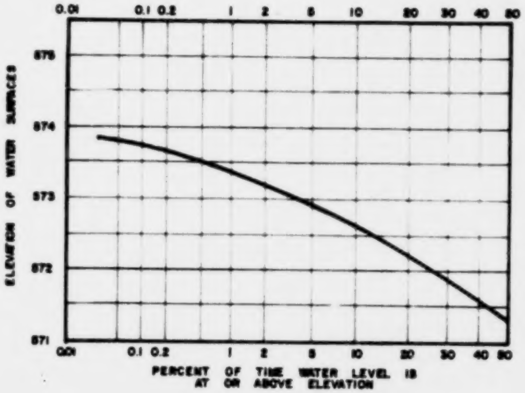
The climate of the Sandusky, Ohio area is influenced somewhat by Lake Erie. Winters are moderately cold; summers are mild and pleasant. Periods of extreme hot and cold weather are of relatively short duration. Differences in elevation are insufficient to cause marked variations within the area. Table 4 gives the normals, means, and extremes for the Sandusky WB Station.



KEY: LEVELS ————
AVERAGE PERIOD OF RECORD ··········
AVERAGE LAST TEN YEARS - - - - -

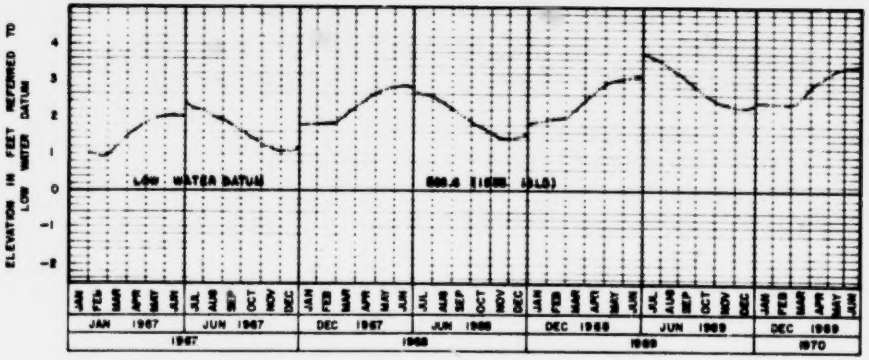
LAKE ERIE LEVELS

FIGURE 4C



LAKE ERIE WATER LEVEL
BASED ON MEAN MONTHLY LEVEL
1860 THRU 1969
BY CITY OF SANDUSKY, OHIO

FIGURE 4A



LAKE ERIE PROJECTED LEVELS

FIGURE 4B

NOTE:
FIGURE B & C ARE BASED ON THE
"MONTHLY BULLETIN OF LAKE LEVELS"
DEPARTMENT OF THE ARMY, LAKE
SURVEY DISTRICT, CORPS. OF ENGINEERS

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AKRON, OHIO

FIGURE 4

TABLE 4
SANDUSKY, OHIO
Normals, Means, and Extremes

	Temperature							WIND	
	Normal			Extremes					
	Daily Max.	Daily Min.	Monthly	High	Year	Low	Year	Mean Hourly Speed	Prevailing Direction
January	35.7	21.8	27.5	73	1950	-16	1879	10.8	SW
February	36.4	22.4	28.1	72	1944	-15	1899	10.9	SW
March	45.2	29.8	36.6	85	1910	- 3	1885	11.3	SW
April	56.3	39.4	47.3	90	1942	14	1923	10.6	SW
May	68.3	50.2	59.2	93	1941	32	1923	9.0	SW
June	79.0	60.7	69.2	104	1934	40	1894	8.1	SW
July	83.7	60.7	69.2	105	1936	50	1918	7.6	SW
August	81.7	63.9	72.3	105	1918	45	1946	7.6	SW
September	75.4	57.5	66.0	99	1953	34	1956	8.4	SW
October	63.5	46.5	54.4	93	1953	22	1925	9.3	SW
November	49.2	35.2	41.7	82	1950	0	1880	10.9	SW
December	37.6	25.3	31.3	70	1899	-13	1880	10.6	SW
Annual	59.3	43.2	50.7	105	July 1936	-16	Jan. 1879	9.6	SW
Years of Record	80	80	83	80	----	80	----	80	80

TABLE 4
SANDUSKY, OHIO
Normals, Means, and Extremes

	Total Precipitation					Snow & Sleet			Mean. No. of Days		
	Normal	Monthly Max.	Year	Monthly Min.	Year	Mean Total	Monthly Max.	Year	Precipitation .01" or more	Snow & Sleet 1.0" or more	Thunderstorm
January	2.28	6.58	1937	0.60	1902	8.3	29.8	1893	14	4	0
February	2.21	8.53	1887	0.27	1920	6.6	22.1	1893	12	3	0
March	2.77	8.69	1913	0.28	1910	4.5	16.1	1916	13	2	2
April	2.81	6.24	1910	0.35	1915	1.2	12.0	1957	12	0	3
May	3.33	9.04	1943	0.64	1934	T	----	----	13	0	5
June	3.79	12.51	1937	0.91	1919	T	----	----	11	0	7
July	3.48	9.71	1943	0.26	1916	----	----	----	10	0	7
August	3.20	8.02	1882	0.23	1894	----	----	----	9	0	5
September	2.77	7.72	1950	0.73	1928	----	----	----	9	0	3
October	2.36	6.22	1917	0.43	1897	T	1.6	1917	10	0	1
November	2.32	6.43	1927	0.09	1904	1.8	12.3	1950	12	1	1
December	2.19	6.27	1881	0.63	1934	6.0	20.2	1951	13	3	0
Annual	33.41	12.51	June 1937	0.09	Nov. 1904	28.4	29.8	Jan. 1893	138	13	34
Years of Record	83	81	----	81	----	77	73	----	81	66	74

The normal annual precipitation for the Sandusky area is 34.01 inches for periods of record of 1931 to 1960. For the 83 years of record, the normal annual precipitation is 33.41 inches. Monthly normals vary from a minimum of 2.19 inches in December to a maximum of 3.79 inches in June. The annual mean temperature is 50.7° Fahrenheit. January is the coldest month with a mean temperature of 27.5° and August is the warmest with a mean temperature of 72.3°.

Prevailing winds are from the southwest and winds exceeding 50 M.P.H. are uncommon. High relative humidity exists throughout the year but the atmosphere is usually not uncomfortable.

Precipitation

A recording rain gage was set up in the drainage area on March 27, 1967. The rain gage was moved to the control building of the pilot demonstration facility on October 18, 1968 to facilitate operation. The rain gage installed incorporated a mechanical weighing mechanism with a bucket reservoir. A twelve-hour continuous recording chart was changed weekly or after each storm event. From March 27, 1967 to November 5, 1969, the rain gage was out of operation for about thirty days total.

The data recorded during the period of operation from November 5, 1969 is summarized in Table 5. Monthly totals of rainfall are given with comparison to normal monthly totals for the 1931 to 1960 period and also to the normal monthly totals for the 1921 to 1950 period for the Sandusky WB Station. Also given are the number of days in which rainfall exceeded 0.01, 0.10, and 0.25 inch.

Table 6 lists the storms which had sufficient rainfall to produce storm overflow from the McEwen Street outfall along with all storm events producing total rainfall or intensities which might produce storm overflow under other conditions. Those storms producing storm overflow caught during the one year of operation are included in Table 9, "Tank Storage Data."

Intensity - duration - frequency curves are included in Figure 5. They are based from excessive short duration rainfall from Sandusky WB Station data. Curves for recurrance intervals of 1,2,5, and 10 years for durations of 5 to 180 minutes are included.

State Climatologist, Mr. Marvin E. Miller, has studied precipitation probabilities for selected locations in Ohio.

TABLE 5
Precipitation
Sandusky Pilot Facility
Foot of McEwen Street
Sandusky, Ohio

Year of Operation		Normal for Month	
		1931-1960	1921-1950
Date		Rainfall In Inches	
5 Nov. - 30 Nov., 1968	2.94*	----	----
Dec. 1968	3.02	2.06	2.16
Jan., 1969	3.83	2.40	2.29
February	0.37	2.09	1.92
March	1.69	2.84	2.89
April	3.23	3.15	2.96
May	3.83	3.52	3.32
June	4.46	4.10	3.73
July	8.21	3.53	3.43
August	0.81	3.27	2.81
September	2.96	2.77	3.26
October	1.99	2.05	2.10
Nov. 1 to Nov. 5, 1969	0.15*	----	----
*November	3.09	2.23	2.27
Total	37.49	34.01	33.16

Number of Days Precipitation Greater Than

Inches	Nov. 5-30	1968 Dec.	1969 Jan.	Feb.	Mar.	Apr.	May
0.01	15	11	12	2	6	14	10
0.10	11	5	6	2	3	10	7
0.25	7	2	4	0	2	5	4

Inches	1969 June	July	Aug.	Sept.	Oct.	Nov.	Total
0.01	17	10	4	10	9	0	147
0.10	13	6	2	6	6	0	77
0.25	5	2	2	5	1	0	39

TABLE 6
STORM EVENTS
Sandusky Pilot Facility
Sandusky, Ohio
November 5, 1968 to November 5, 1969

Storm Overflow Produced	Date	Precipitation (Inches)	Average Intensity (In/Hr.)	Comments
	1968			
	Nov. 15	0.65	0.08	Rainfall;No Overflow
	28	0.59	0.17	Rainfall;No Overflow
1.	Dec. 27 & 28	1.84	0.15	Rainfall
	1969			
	Jan. 18	0.82	0.08	Rainfall;No Overflow
	28 to 30	2.07	0.06	Rainfall;No Overflow
	Feb.			No Overflow
	March 24	0.59	0.06	No Overflow
	April 1	0.53	0.07	No Overflow
2.	18 & 19	1.80	0.12	
	May 7	0.29	1.74	No Overflow
3.	10 & 11	0.89	0.09	
	17	0.24	0.24	No Overflow

TABLE 6
STORM EVENTS
Sandusky Pilot Facility
Sandusky, Ohio
November 5, 1968 to November 5, 1969

Storm Overflow Produced	Date	Precipitation (Inches)	Average Intensity (In/Hr.)	Comments
4.	18	1.68	0.17	
	June 1	0.32	0.96	No Overflow
5.	2	0.60	0.15	
	14	0.24	0.12	No Overflow
6.	15	0.82	0.08	
7.	19	1.25	0.83	
	22	0.24	0.24	No Overflow
8.	23	0.41	----	
9.	July 4 & 5	5.57	0.70	Severe Flooding
10.	17	1.61	0.59	
	Aug. 17	0.50	0.33	No Overflow
11.**	Sept. 6	0.74	0.74	
12.	16 & 17	1.36	0.19	Slight Overflow
	Oct. 31	0.84	0.80	No Overflow

Each storm event listed is considered a separate event only if 12 hours separate it from a prior rainfall.

** Theoretical Overflow - Data acquisition prevented due to malfunctioning of flowmeters. Insufficient flow for storage.

Data contained in Research Bulletin 1017, "Monthly and Annual Precipitation Probabilities for Selected Locations in Ohio," by Mr. Miller and Mr. G.R. Weaver published in March, 1969 by the Ohio Agricultural Research and Development Center has been incorporated in Figures 6 and 7. Figure 6, "Monthly Precipitation Probabilities for Sandusky, Ohio," and Figure 7, "Yearly Precipitation Probabilities for Sandusky, Ohio" are based on Sandusky WB data from 1936 to 1965. The data was prepared using a computer program utilizing the incomplete gamma distribution to calculate precipitation amounts for the selected probability levels.

Further work by Mr. Miller, as yet unpublished, is incorporated in Figure 8, "Monthly Maximum Daily Precipitation for Return Periods in Years for Sandusky, Ohio," and Figure 9 "Annual Maximum Daily Precipitation for Return Periods in Years for Sandusky, Ohio."

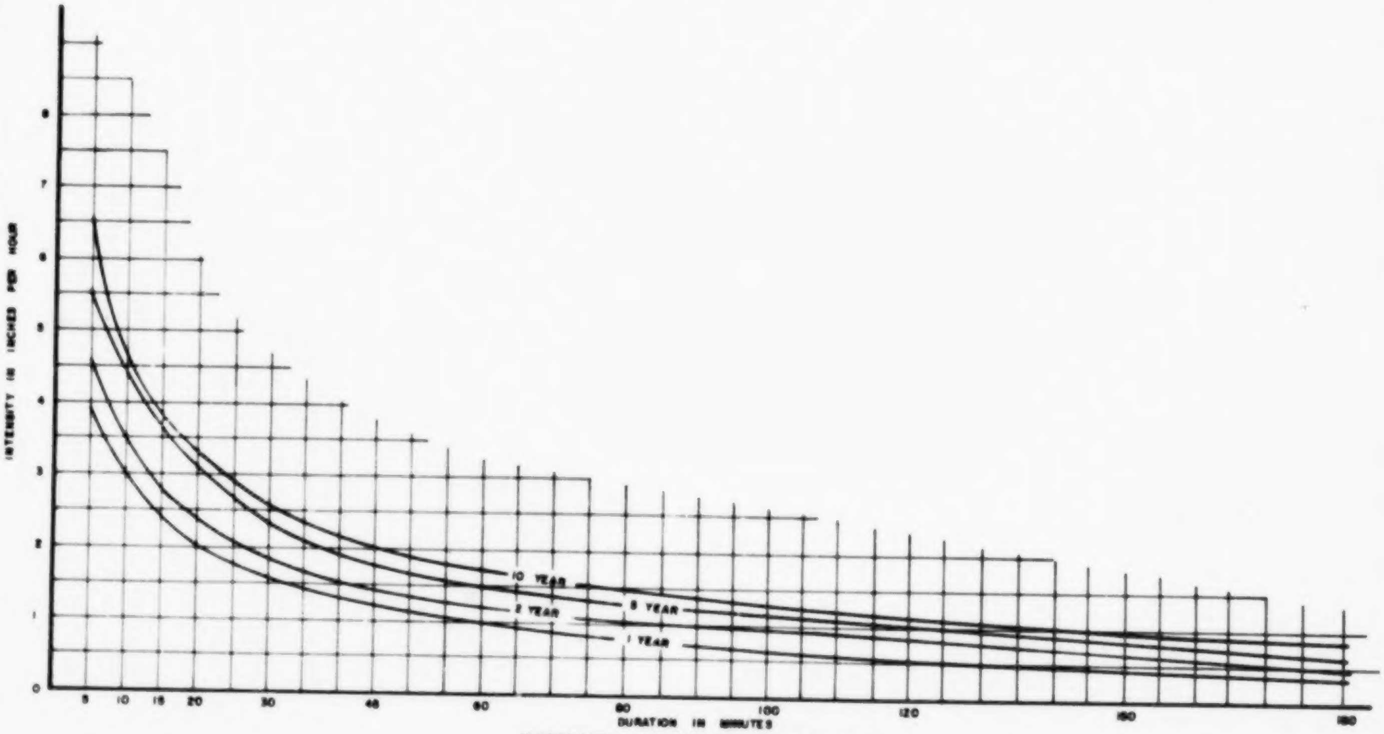
Drainage Area

The McEwen Street, Sandusky, Ohio combined sewer drainage area selected for the pilot demonstration facility is located east of the main business district. The total drainage area is shown in Figure 10. It has an area of 14.86 acres. Figure 11 is an aerial photograph of the area taken on May 3, 1967.

The drainage area is mainly an older residential section. One commercial establishment, a restaurant, is in the drainage area. A public elementary school is located on the southeastern edge of the drainage area but all sanitary sewage is pumped to the Arthur Street drainage area along with storm water from roof drains and the parking lot. Sanitary sewage is collected from 77 separate structures. There are 82 dwelling units in the 76 residential structures.

According to a housing studies report completed by the Erie Regional Planning Commission in March, 1965, for the City of Sandusky, the external condition of the residental structures in the blocks containing the drainage area are rated: good, 61%; fair, 32%; poor, 5%; critical, 2%. Five older residences have been converted to multiple dwelling units. Four of these have two dwelling units and one has three.

The McEwen Street drainage area sewerage consists of vitrified clay pipe 6 to 12 inches in diameter draining to the 30-inch brick sewer in McEwen Street and First Street. Figure 12 shows the combined sewers and manhole inverts in the McEwen Street drainage area.



INTENSITY – DURATION – FREQUENCY
(EXCESSIVE SHORT DURATION RAINFALL)
SANDUSKY, OHIO 1943 – 1964

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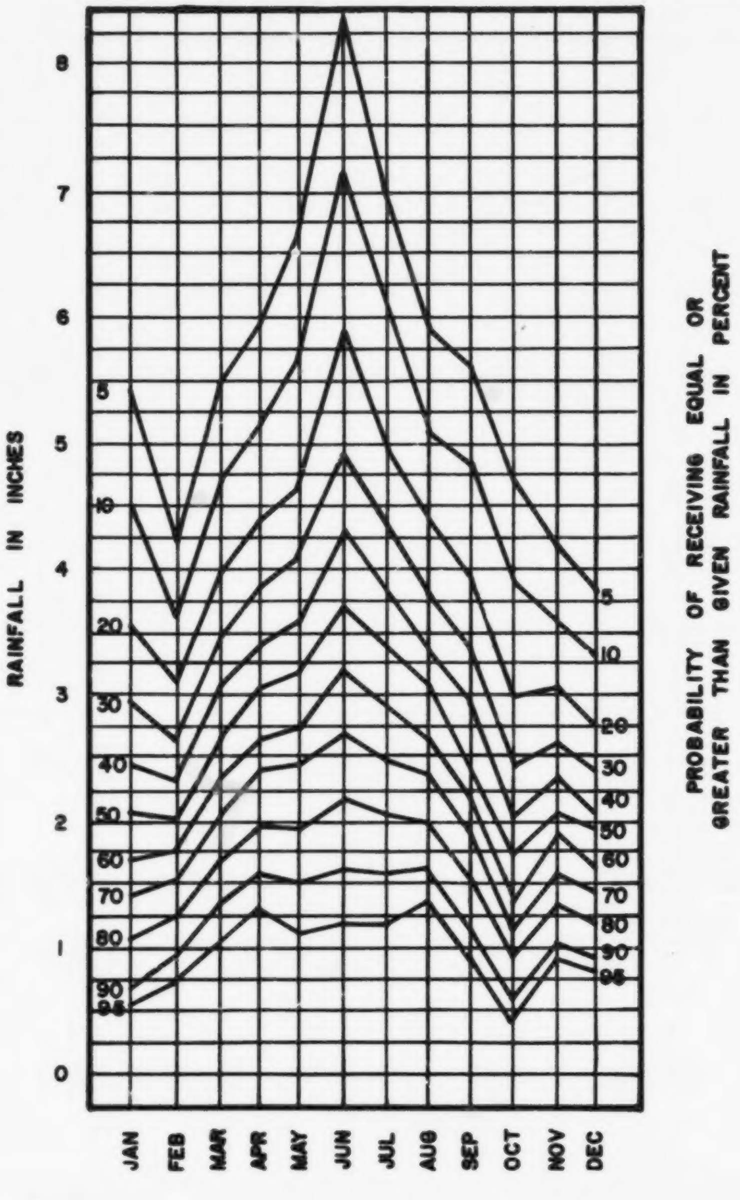
AKRON, OHIO

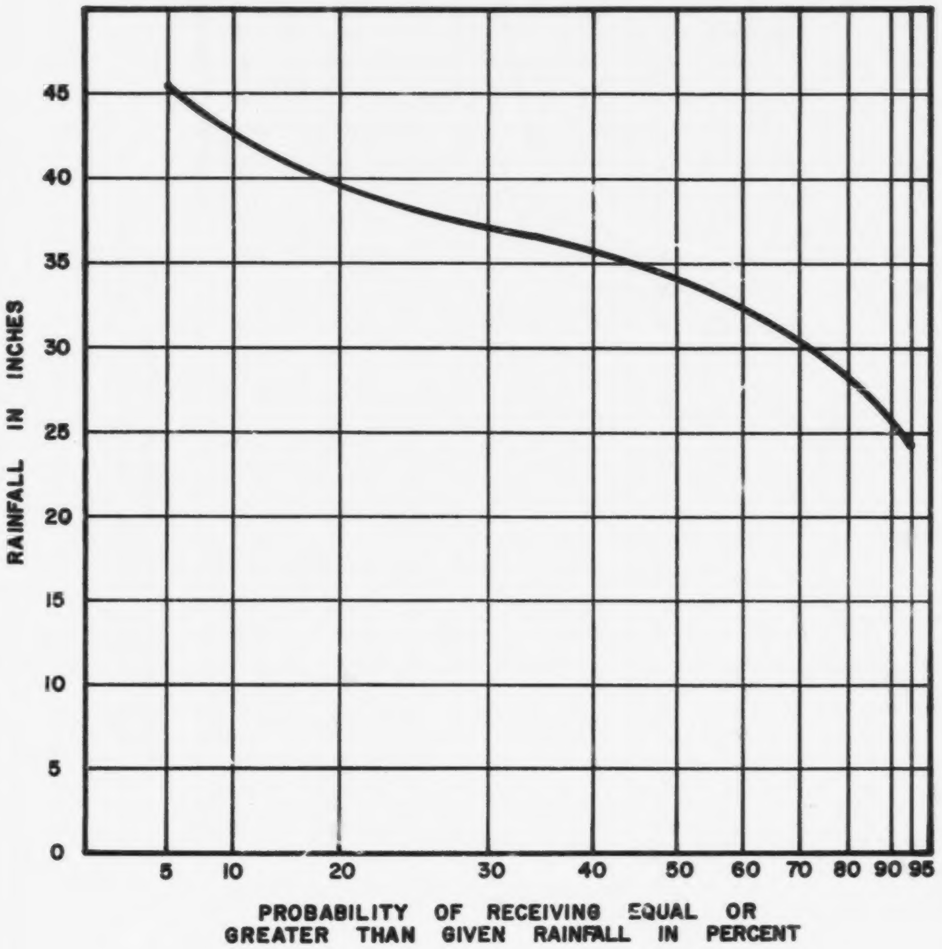
FIGURE 5



MONTHLY PRECIPITATION PROBABILITIES
SANDUSKY, OHIO

KARL R. ROHRER ASSOCIATES, INC.
AKRON, OHIO

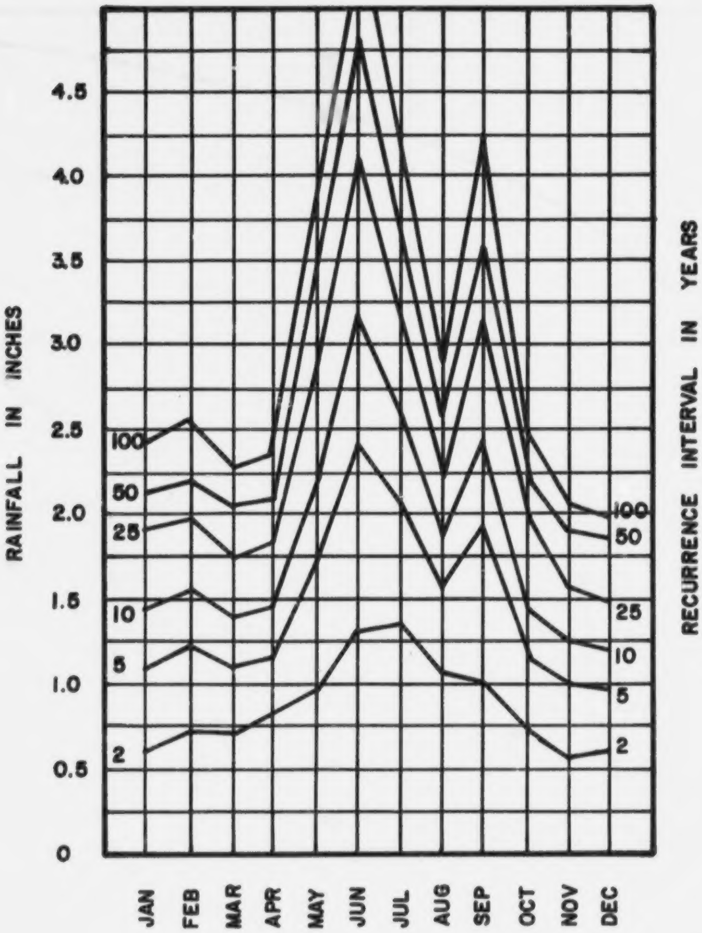
FIGURE 6



YEARLY PRECIPITATION PROBABILITY
SANDUSKY, OHIO

KARL R. ROHRER ASSOCIATES, INC.
AKRON, OHIO

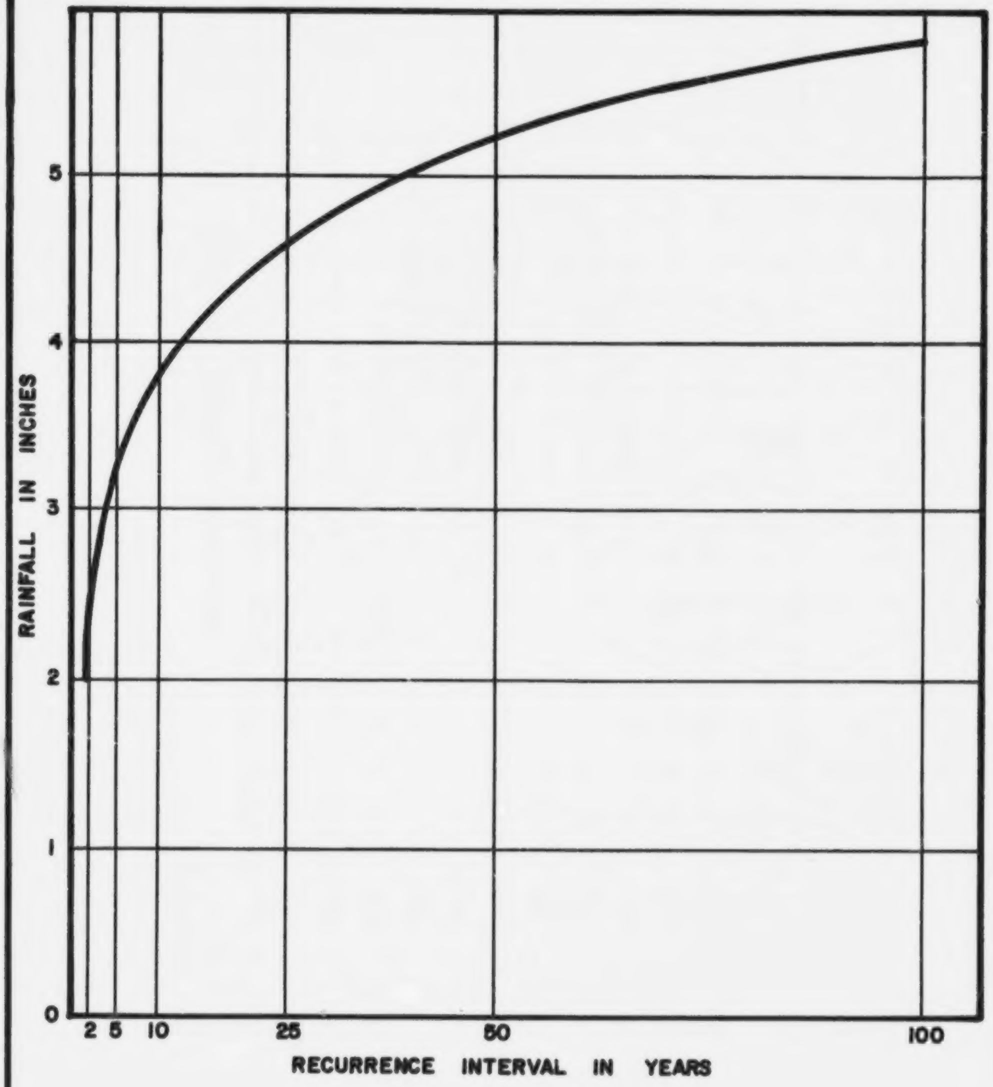
FIGURE 7



MONTHLY MAXIMUM DAILY PRECIPITATION
SANDUSKY, OHIO

KARL R. ROHRER ASSOCIATES, INC.
AKRON, OHIO

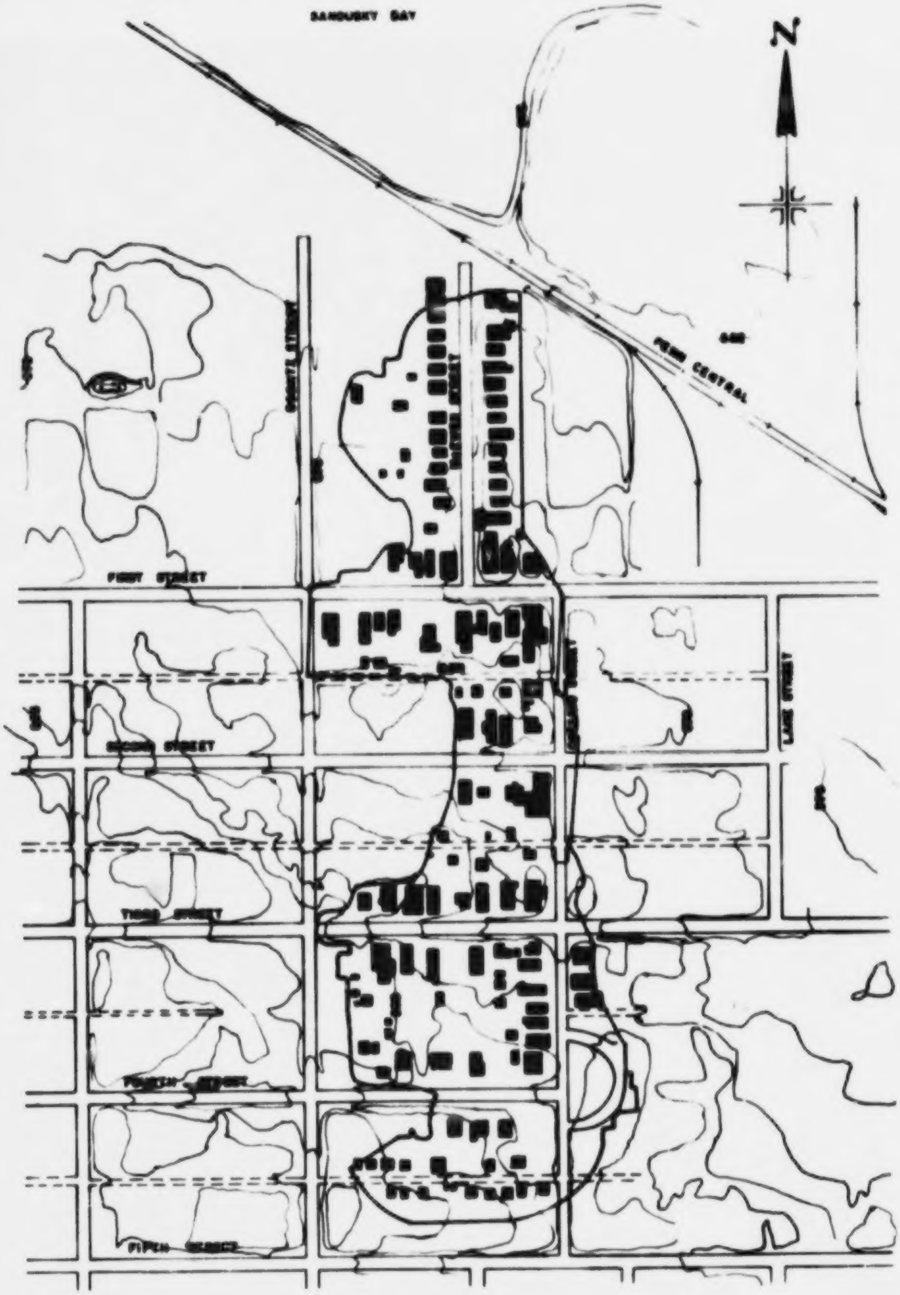
FIGURE 8



ANNUAL MAXIMUM DAILY PRECIPITATION
SANDUSKY, OHIO

KARL R. **ROHRER** ASSOCIATES, INC.
AKRON, OHIO

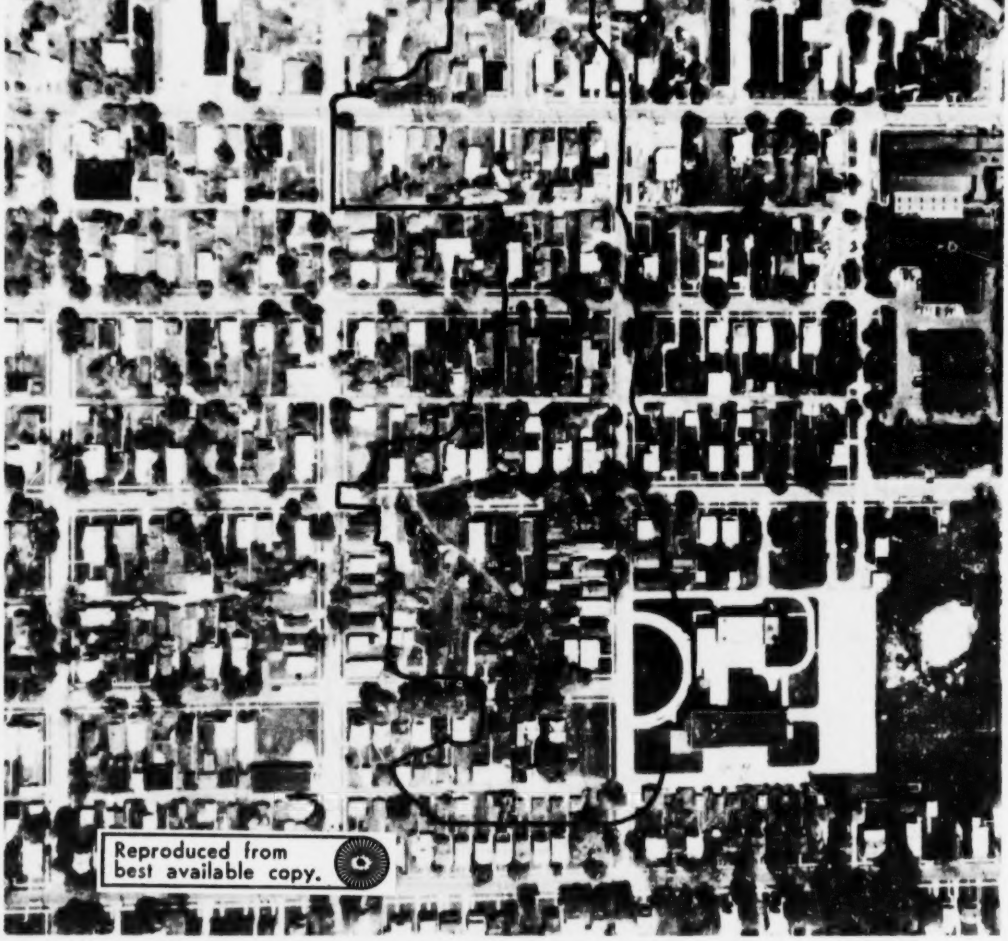
FIGURE 9


SANDUSKY BAY
N
PENN CENTRAL
FIRST STREET
SECOND STREET
THIRD STREET
FOURTH STREET
FIFTH STREET
LAKE STREET

McEWEN STREET DRAINAGE AREA

KARL R. ROHRER ASSOCIATES INC.
AKRON, OHIO

FIGURE 10



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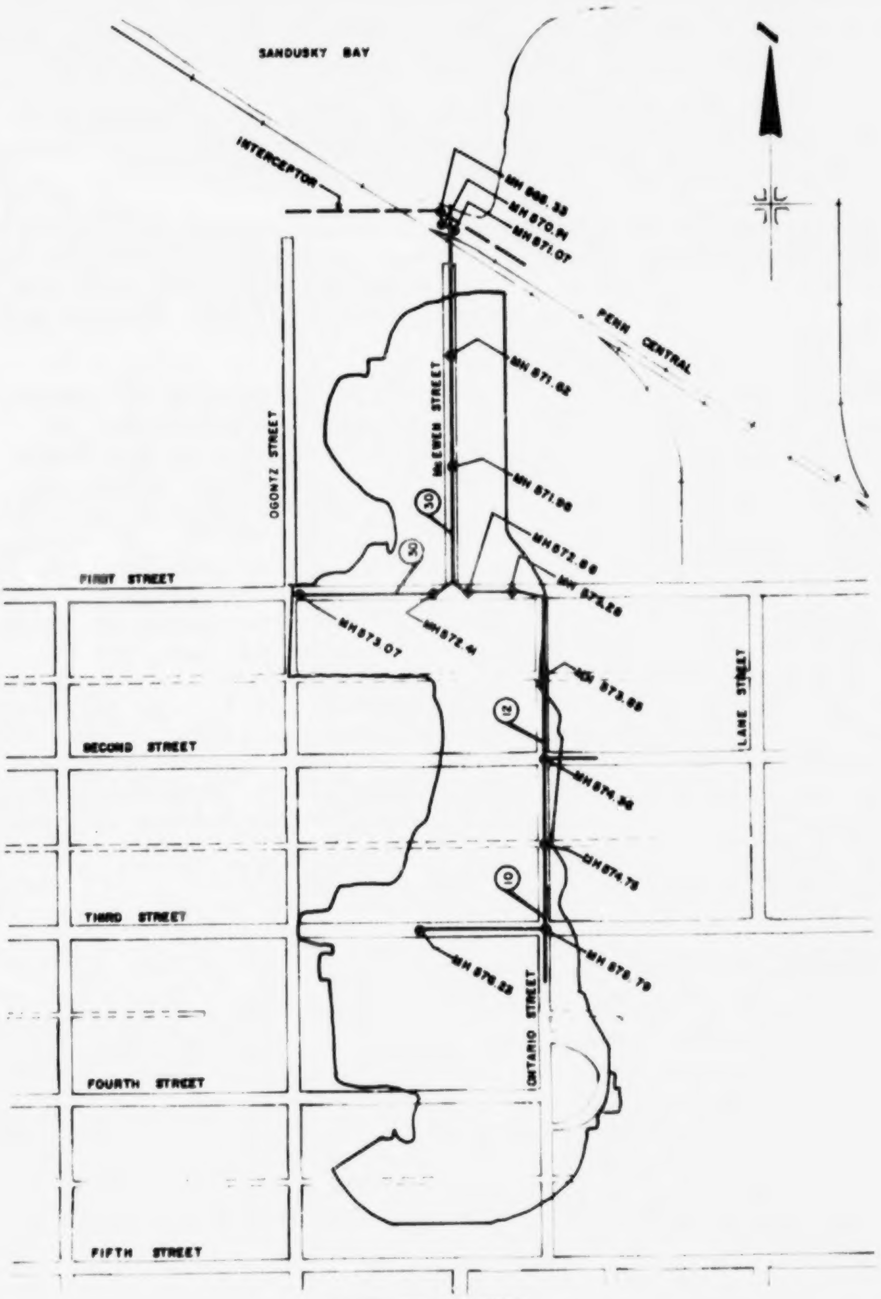
McEwen Street Drainage Area Figure 11


SANDUSKY BAY
INTERCEPTOR
MH 868.33
MH 870.91
MH 871.07
PENN CENTRAL
MH 871.62
McEWEN STREET
OGONTZ STREET
30
MH 871.98
50
MH 872.88
MH 873.20
FIRST STREET
MH 873.07
MH 872.41
LANE STREET
MH 873.68
12
SECOND STREET
MH 874.36
MH 874.78
10
THIRD STREET
MH 876.23
MH 875.79
ONTARIO STREET
FOURTH STREET
FIFTH STREET

COMBINED SEWERS
McEWEN STREET DRAINAGE AREA

KARL R ROHRER ASSOCIATES INC
AKRON, OHIO

FIGURE 12

The combined sewer collection system can handle existing dry-weather flow but is badly undersized for even moderate storm events due to the flat slopes in the drainage area and the capacity of the 6-inch vitrified clay sewers.

The soil in the drainage area consists of a Lacustrine silty clay loam with a moderately slow rate of permeability. Many trees line the streets in the drainage area.

The curb inlets to the combined sewer system are located on paved streets. Alleys between Ogontz and Ontario Streets are unpaved. Due to the low soil permeability and negligible ground slope, storm water does not drain freely but backs up in yards of residents.

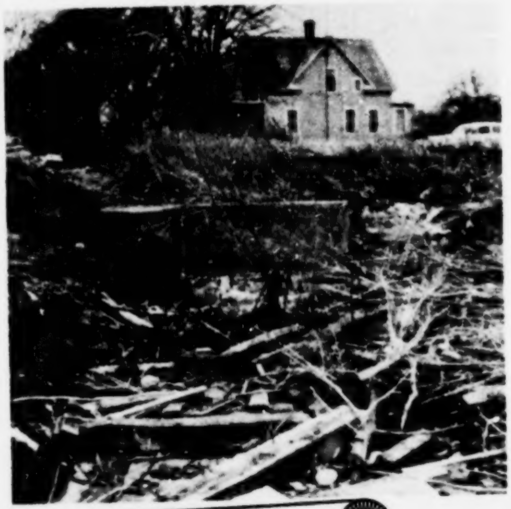
During preliminary investigations, the amount of runoff from the McEwen Street drainage area was studied. A weighted runoff coefficient was determined to be about 0.42. The original design storm, a one year storm of 60 minute duration, with a 1.30 in/hr estimated intensity yielded a total of 220,000 gallons of runoff. However, due to the flat slopes, the undersized 6-inch sewers, the surface depression storage and the relatively large amount of vegetation, a lower runoff coefficient would be more probable for the design storm. Checks of the runoff coefficient for the McEwen Street drainage area during the operation period were not successful due to the poor operation of the flowmeters at the pilot facility. Data to construct curves showing the pollution control of the pilot facility to intensity - duration - frequency of storm events and the pollution control to frequencies of storm water overflow peaks and volumes was not acquired for this same reason.

McEwen Street Regulator - Outfall

The Mc Ewen Street 30-inch brick sewer brings flows from the McEwen Street drainage area to the regulator - overflow device at the foot of McEwen Street. The regulator consists of an adjustable leaping weir in an underground concrete chamber with a second underground chamber accepting flow dropping through the weir and diverting it to the interceptor sewer. Any flow leaping the weir passes through a 30-inch reinforced concrete outfall conduit to an outfall at the shore of Sandusky Bay.

A non-sealing gate was positioned over this outfall shown on Figure 13, to prevent debris from being washed back into the outfall conduit. No effort was made to prevent bay water from flowing into the outfall.

Theoretically the overflow water passed to the end of the 30-inch outfall pipe where a drop inlet to a 12-inch corrugated steel pipe would carry the overflow about 250 feet out into the bay. The drop inlet structure was completely filled with grit.



McEwen Street
Outfall

Figure 13

Reproduced from best available copy.

The McEwen Street leaping weir had a theoretical interception ratio of 91 prior to May 20, 1969, when it had an opening width of six inches. To increase the frequency of storm overflow from the McEwen Street drainage area during the year of operation, the leaping weir was closed to a two-inch width with a corresponding theoretical interception ratio of about 12.5 in May, 1969.

The high interception ratio is misleading due to the small drainage area of 14.86 acres which is tributary to the McEwen Street regulator. The average dry-weather flow is only 0.026 cfs. Wet-weather flow required to result in overflow to storage was 2.36 cfs. prior to May 20, 1969 and 0.33 cfs. afterwards. Capacity of the McEwen Street 30 inch diameter brick combined sewer is about 19.5 cfs. maximum.

Bedrock Elevation

From conversations with city employees supervising in the construction of the present interceptor sewer to the treatment plant, a problem of location of bedrock had been established; however, bedrock had not been encountered at the McEwen Street outfall during that construction. Interceptor invert is about 569.2 at this point. Proposed tank bottom elevation was to be about 562.9.

Boring reports were acquired from the Farrell-Cheek Steel Company dated April, 1964. Three test holes had been bored by Herron Testing Laboratories, Inc., Cleveland, Ohio for a plant expansion about 360 feet east of the McEwen Street outfall. Bedrock was encountered consistently at 20 feet below the surface elevation of approximately 582. This showed that bedrock elevations might begin at 562 which would be below the designed tank bottom elevations by about one foot.

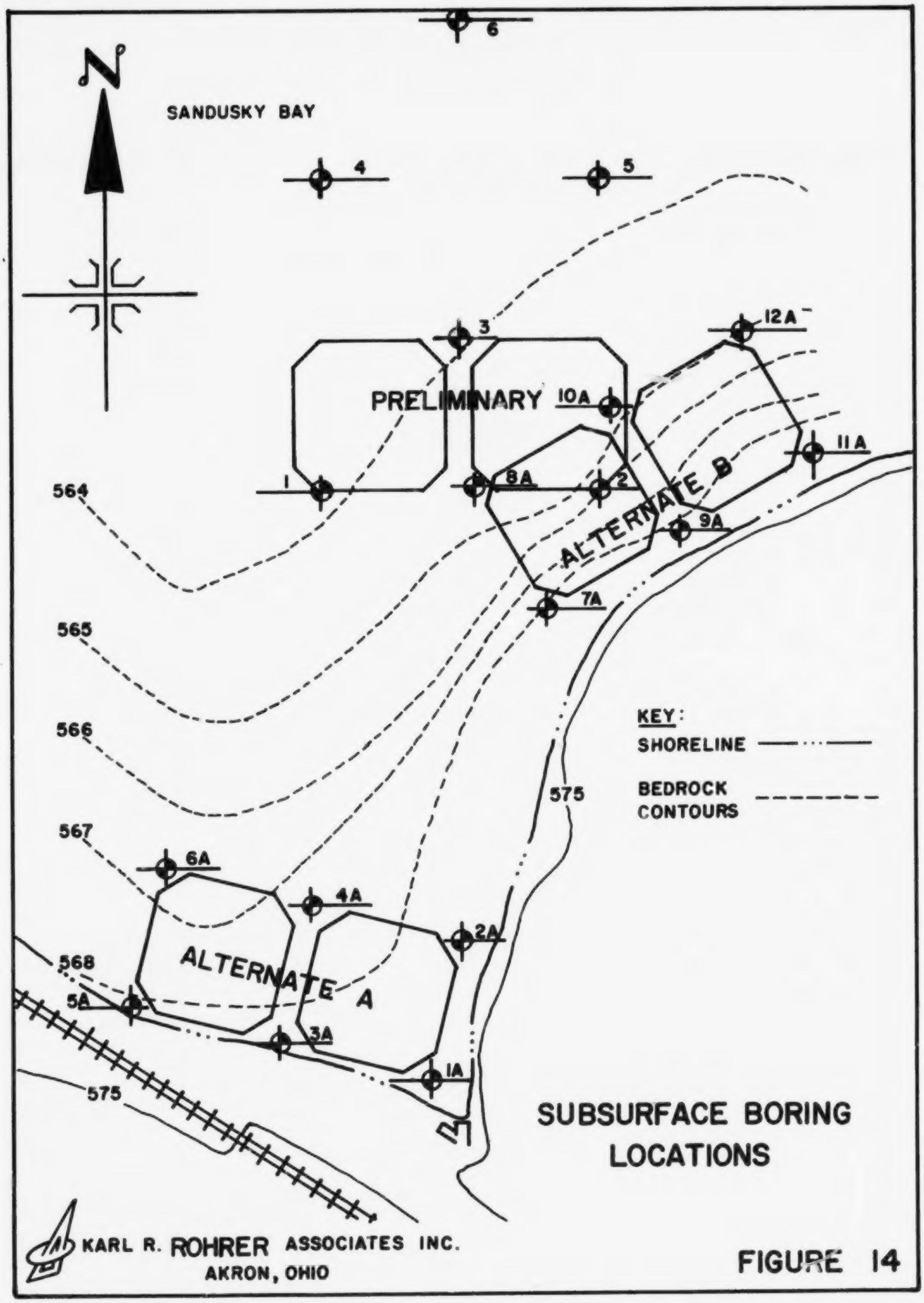
During preliminary site investigations conducted under F.W. P.C.A. Contract No. 14-12-25, requests were made by Rohrer to obtain test borings at the anticipated location of the proposed underwater storage tanks. These requests were turned down by the F.W.P.C.A. until after the F.W.P.C.A. Contract No. 14-12-143 was negotiated.

After F.W.P.C.A. Contract No. 14-12-143 was in effect, six test holes were drilled by Herron Testing Laboratories, Inc. on January 19,20, and 24, 1968. The test holes were located at the preliminary tank location as shown on Figure 14. Bay water sounding showed that water depth increased with the distance from shore to approximately 5 feet at 500 feet north of McEwen Street Outfall.

The test holes were drilled through 12 to 18 inches of ice on the bay. Test holes 1, 3, 4 and 5 were driven to refusal. Test holes 2 and 6 were drilled five feet into the limestone bedrock. The results of these test borings in locating the bedrock and the unconfirmed compressive strength of the grey limestone is given in Table 7.

The higher bedrock elevation encountered required that further investigations be conducted at two alternate tank locations. The rock excavation required to deepend the primary tank location to sufficient depth would have increased system costs excessively due to the wet installation methods that would have had to be used.

Rohrer conducted test boring to help determine the two alternate tank locations using a portable drilling kit. A total of six test borings were made on February 14 and 15, 1968.


N
SANDUSKY BAY
6
4
5
3
12A
PRELIMINARY
10A
11A
1
8A
2
ALTERNATE B
9A
7A
564
565
566
567
568
575
575
KEY:
SHORELINE
BEDROCK CONTOURS
6A
4A
2A
ALTERNATE A
5A
3A
1A

SUBSURFACE BORING LOCATIONS

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FIGURE 14

TABLE 7
Test Borings

Hole No.	Date	Depth of Refusal	Bedrock Elevation
		Preliminary	
1	1-19-68	5' - 0"	569.5
2	1-20-68	6' - 0" (Weathered to 11' - 0")	568.6 @ 11'-0"
3	1-19-68	6' - 0"	568.5
4	1-24-68	6' - 6"	568.0
5	1-24-68	5' - 0"	569.5
6	1-19-68	5' - 10"	568.7
		Alternate A	
1A	3-4-68	7' - 3"	567.2
2A	3-4-68	6' - 2"	568.3
3A	3-4-68	8' - 2"	566.3
4A	3-4-68	5' - 4"	569.2
5A	3-4-68	7' - 3"	567.2
6A	3-4-68	5' - 6"	569.0
		Alternate B	
7A	3-4-68	6' - 6"	568.0
8A	3-4-68	5' - 6"	568.8
9A	3-1-68	6' - 8"	567.8
10A	3-1-68	7' - 1"	567.4
11A	3-1-68	3' - 5"	571.1
12A	3-1-68	6' - 10"	567.7

Unconfined Compressive Strength

Hole No.	Date Drilled	Date Tested	Depth	Unconfined Compressive Strength (Ton/Sq. Ft.)
2	1-20-68	1-23-68	7' - 0"	865.9
2	1-20-68	1-23-68	13' - 0"	830.9
6	1-19-68	1-23-68	6' - 0"	865.9
6	1-19-68	1-23-68	10' - 0"	580.5

The results showed that bedrock elevations were encountered at 566.3 feet 600 feet north of the McEwen Street outfall headwall and at about 564.0 feet 50 feet north. Alternate A location showed bedrock at about 566.3 and alternate B at 566.1.

On March 1 and 4, 1968, Herron Testings Laboratories, Inc. bored twelve additional test holes. See Figure 14. The results are shown on Table 7.

From the March, 1968 borings, the alternate tank location A was chosen. The choice was based on lowest bedrock elevations, ease in construction, and ability to get the required permits for construction.

SECTION V

SYSTEM DESIGN

The system proposed to the F.W.P.C.A. for underwater storage of storm overflow from a combined sewer by Rohrer consisted of three basic system components, the underwater storage tank with its associated piping and controls, a connecting structure to the existing outfall, and a control building to house instrumentation and pump system. With receipt of the contract for the development of the preliminary design for the underwater storage system on December 27, 1966, several general system goals were set. These consisted of:

1. Gravity fill with fully automatic operation,
2. Minimum pre-treatment of overflow preceeding the storage tank,
3. Standard construction and system installation,
4. Minimum maintenance and operator requirements, and
5. Minimum operation cost and lowest possible capital costs.

Analysis of the storage system and the environment in which it was to work pointed out several inherent problems.

In general, these were:

1. Continuous monitoring of tank volume at all times.
2. Control and prediction of tank shape while filling in the underwater environment.
3. Material selection and best tank shape.
4. Control of undissolved solids and removal from tank with minimum pretreatment prior to tank, and
5. Control of entrained and evolved gases in stored liquid.

General design parameters were controlled by the system itself, an underwater storage system for the off-shore temporary storage of storm overflow from a combined sewer in flexible tanks, the chemical and physical properties of the liquid to be stored, and the physical restraints of the site.

The site selected had several design limiting factors. The water body, Sandusky Bay, is shallow, there is bedrock near the surface of the bay bottom, little hydraulic head is available to allow the necessary gravity fill of the tanks, and no restriction of overflow could be tolerated due to flat gradients in the sewer collection system in the drainage area.

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Model Studies

Included in the proposal to the F.W.P.C.A was laboratory testing of tank models and model tank components to assist in providing a practical tank design consistent with design parameters and to provide the best practical solution to inherent system problems.

The laboratory testing of the models produced practical solutions for:

1. Tank shape,
2. Removal and control of undissolved solids in the stored liquid, and
3. Control of evolved and entrained gases.

During February and March, 1967, the equipment necessary for hydraulic testing was installed at the Rohrer facilities. This consisted of:

1. A 32 foot by 16 foot by 3.5 foot vinyl lined test tank with a work platform at one end and a movable bridge spanning the test tank,
2. A 625 gallon supply tank to provide water to model tanks at varying hydraulic heads,
3. A device to measure the static hydraulic head imposed, and
4. Two centrifugal pumps and one high pressure pump with the necessary fittings and PVC pipe as required.

Storage Tank Evolution

The liquid to be stored in the facility was combined sewer overflow. The total tank storage capacity was limited by the F.W.P.C.A. to approximately 200,000 gallons. From preliminary design figures, this volume would intercept all runoff from the McEwen Street drainage area for a one-year storm of sixty minute duration.

The first area to be determined was the best general tank shape. Originally proposed, the storage tank would incorporate a "pillow" type storage tank made of compatible materials; and, a supporting structure would be designed for this "pillow" type tank.

The "pillow" type tank as the basic component of the storage tank allows one of the most economical approaches for tank costs. These "pillow" type tanks are commercially available "off-the-shelf" items available from rubber oriented companies.

However, since the liquid to be stored has a specific gravity equal to the liquid into which the tank is to be immersed, a problem exists in controlling the tank shape and predicting this shape. From model studies conducted, the "pillow" type tank tends to take a cylindrical shape when filled and no particular shape on filling and emptying in its underwater environment. The liquid could not be pumped out. When the stored liquid near the effluent port was removed, the effluent port became clogged with the flexible tank membrane. Since no difference between the two liquid densities exists, buoyancy forces could not be used for effective stored liquid removal as is done in the underwater storage of oil and certain other products.

A support system required for the "pillow" type tank also creates considerable wearing of the tank membrane where wave action exists. Since an extensive support system of webbing would be required to hold the "pillow" type tank in the shape desired, an alternative tank shape was desirable.

Testing of the "pillow" type tank also showed that removal of entrained or evolved gases in this tank would be difficult. The support structure of nylon webbing would provide a general tank shape but would not allow removal of gases from one common point. An extensive number of gas vents would be necessary.

The ability to predict tank shape was considered to be necessary, since continuous monitoring of tank volume could be most easily related to tank height, internal tank pressure, or some other tank parameter.

One last problem presented itself from the model tests conducted. This was, that in a shallow water condition with wave action, the resonant frequency of a filled or partially filled tank might possibly be equalled with the rectangular "pillow" type tank. Fabric stress could then be exceeded and tank failure could result.

After testing the "pillow" type tank model, three separate models of a subsequent design were tested. This tank design consisted of a steel pipe frame in the form of a modified octagon with a flexible membrane top and bottom. The tank top was to conform to the bottom contours when empty and the top would rise on filling. The anchoring of the tank would be accomplished by use of the steel frame and the membranes would be clamped to the frame for the tank top and bottom.

General studies were conducted on the different models of the "pillow" type tank and the second general tank design to determine what reduction of flow would occur in the influent pipe to the tank as compared to that of a pipe with no tank on the end. Dr. Andrew Simon, Head of the Department of Civil Engineering at the University of Akron , was consulted on this above comparison as well as other questions in the model studies. From the experiments conducted, Dr. Simon derived a percentage reduction in flow which would be expected to occur in the prototype tank. The Appendix contains Dr. Simon's general report on this study. Due to the small percentage reduction in flow for the flexible underwater tanks, this reduction could be considered negligible for the flows expected to be encountered.

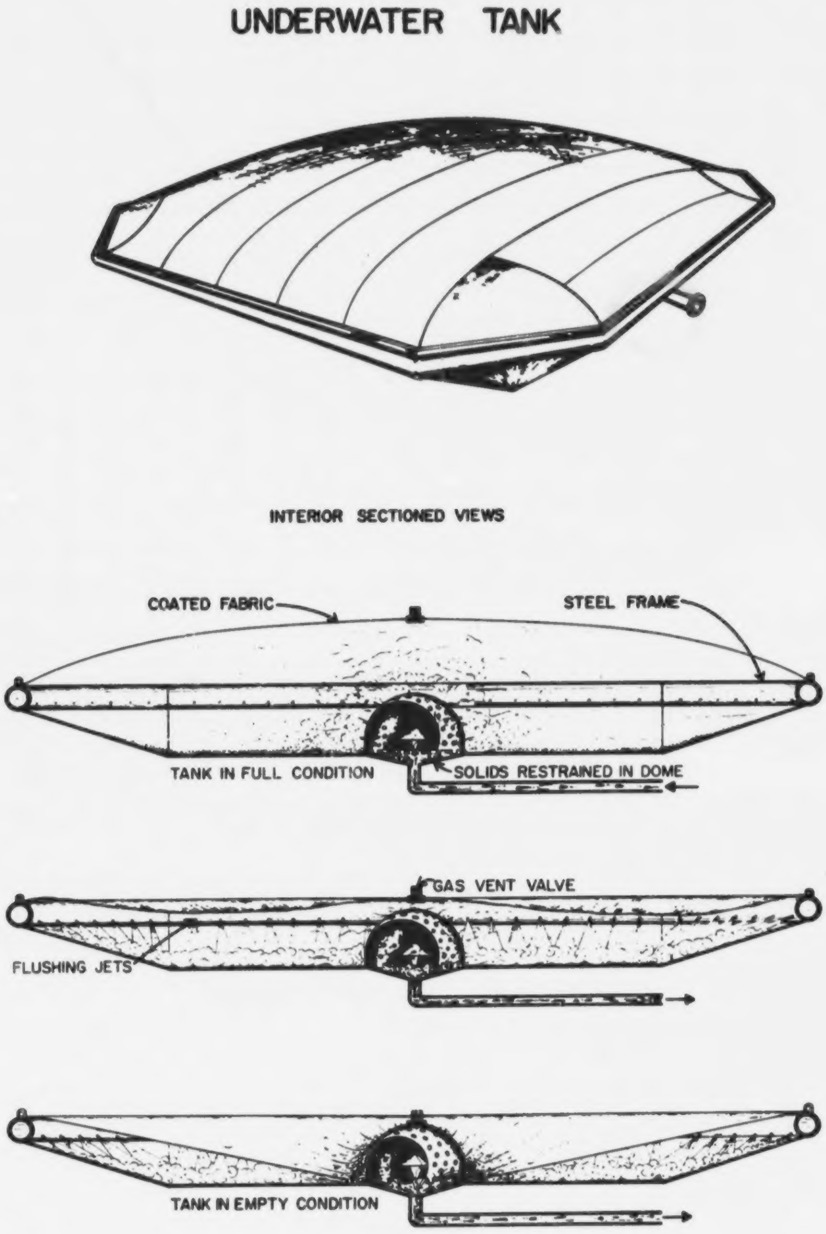
The site conditions of shallow water suggested the use of the second tank design tested due to its lower profile. A low tank profile requires a low internal tank pressure to keep stresses in the top and bottom tank membranes from becoming excessive. A pressure relief valve to prevent excessive internal tank pressures solved this problem.

Flexible Membrane

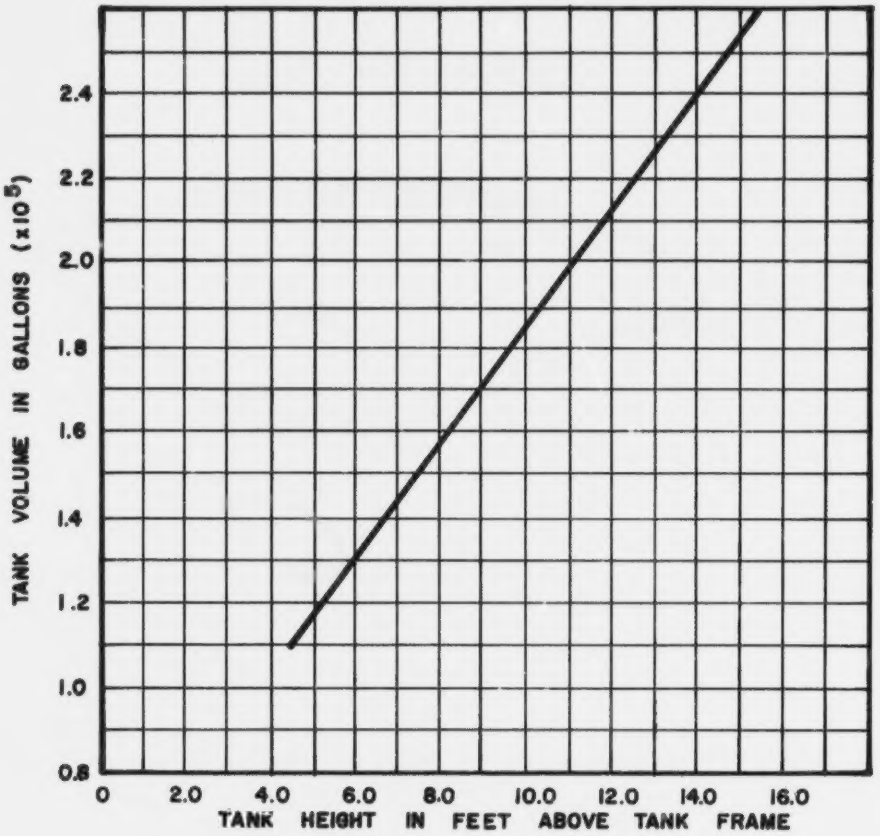
After a comparison study of the materials available to construct the flexible tank top and bottom membranes was completed, a two-ply neoprene rubber coated nylon fabric was selected for the tank membrane due to the underwater environment, the type of liquid to be stored, and the fabric strength requirements. The Appendix contains a description of the chemical and physical properties of the membrane.

From the laboratory testing done, the use of a rigid bottom was suggested. If a rubber top and bottom were used, sediment filling the tank might cause the bottom to fail if no support were given to it. But even more important to the tank design, the bottom membrane in the model tests tended to come up to meet the tank top. As stated before, the tank top was designed to fit the tank bottom contours when empty. It was decided to use two 100,000 gallon tanks allowing a comparison of a steel bottom versus a fabric bottom to find if one was more advantageous than the other. Figure 15 is a rendering of the steel bottom tank design.

The elongation of the fabric was of particular interest during design. With a 6% fabric elongation being possible within stress limitations, the additional fabric length could add about 80,000 gallons to the 100,000 gallons design capacity at the same cost. Figures 16, 17, 18, 19, and 20


UNDERWATER TANK
INTERIOR SECTIONED VIEWS
COATED FABRIC
STEEL FRAME
TANK IN FULL CONDITION
SOLIDS RESTRAINED IN DOME
GAS VENT VALVE
FLUSHING JETS
TANK IN EMPTY CONDITION

Underwater Tank Figure 15



TANK HEIGHT ABOVE TANK FRAME
VERSUS
VOLUME (WEST TANK)

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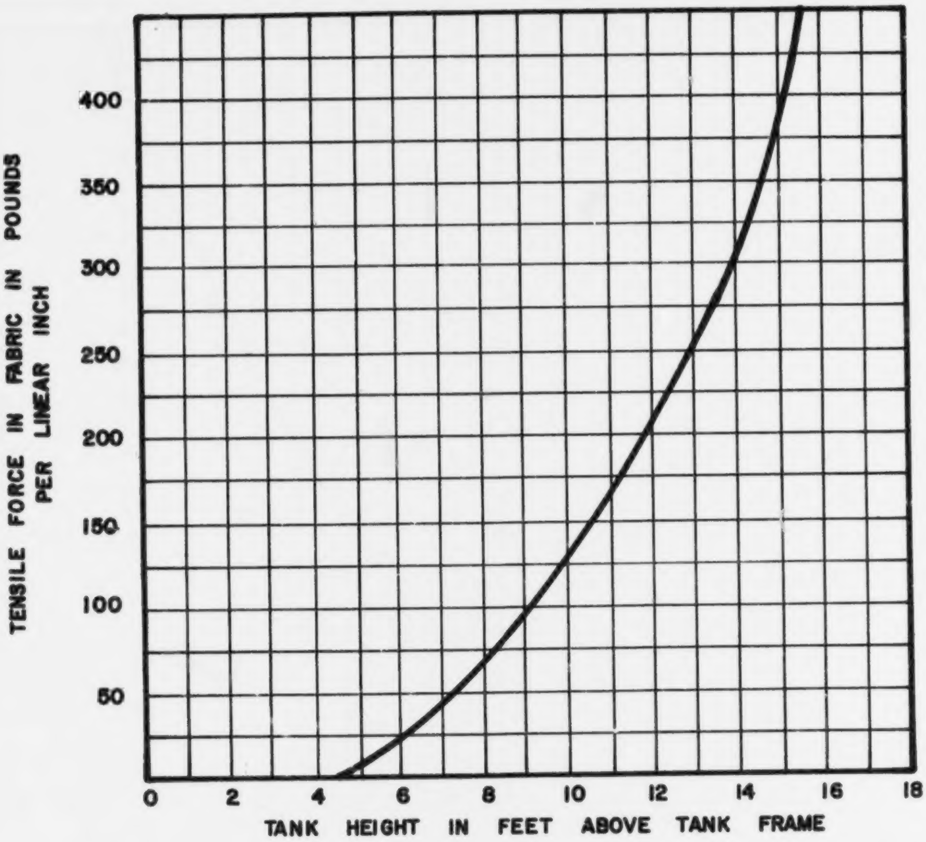
FIGURE 16



TANK HEIGHT ABOVE TANK FRAME
VERSUS
TENSILE FORCE IN FABRIC
(WEST TANK)

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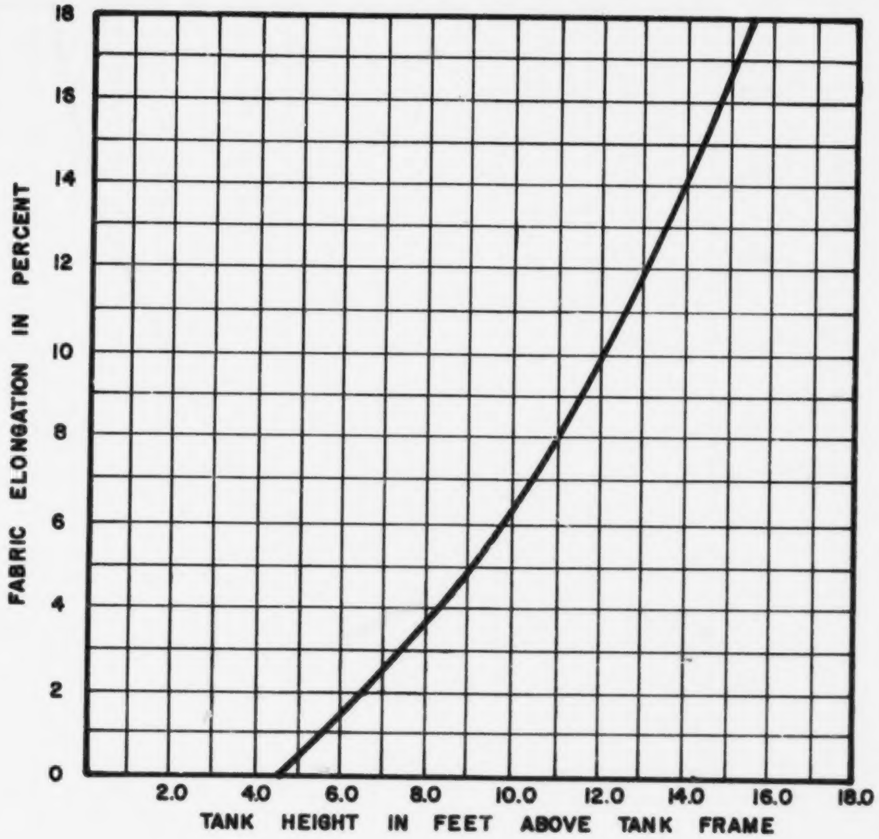
FIGURE 17



TANK HEIGHT ABOVE TANK FRAME
VERSUS
FABRIC ELONGATION (WEST TANK)

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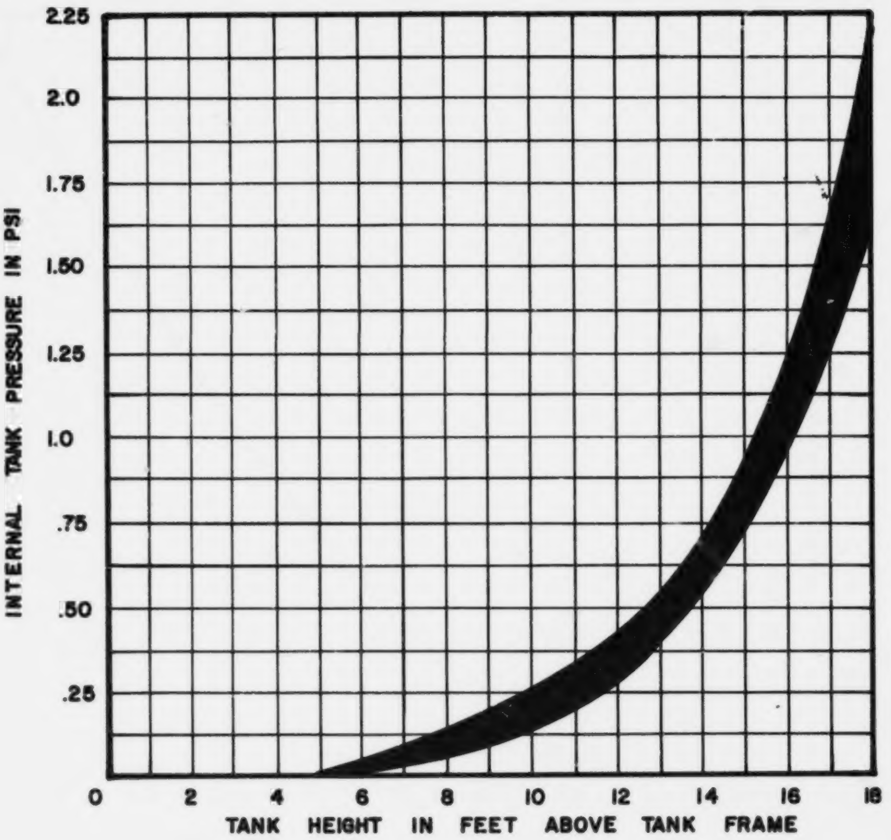
FIGURE 18



TANK HEIGHT ABOVE TANK FRAME
VERSUS
INTERNAL PRESSURE (WEST TANK)

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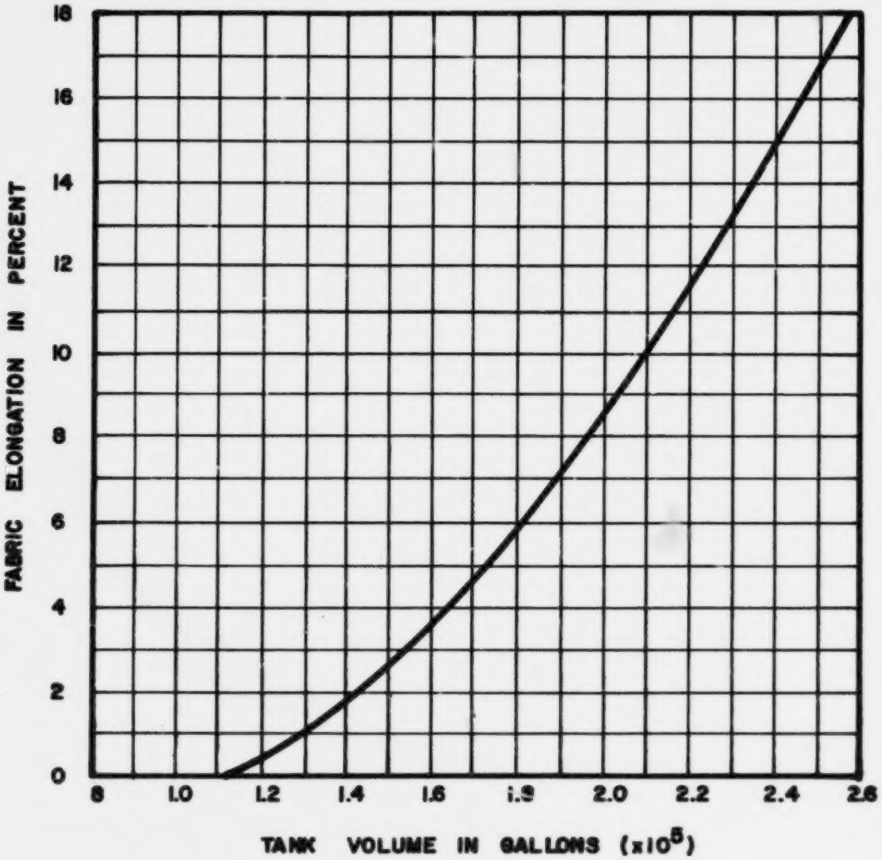
FIGURE 19



TANK VOLUME
VERSUS
FABRIC ELONGATION (WEST TANK)

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AKRON, OHIO

FIGURE 20

show theoretical tank curves for the steel bottom tank installed in Sandusky, Ohio. The design capacity was based on zero fabric elongation due to lack of experience or knowledge of this type of installation by the manufacturer. Working elongation of 6% can be effectively utilized until further data is available.

For tank design contemplated, a method of attaching the fabric to the tank frame had to be designed. Since no existing standard clamping method could be used to the strength required and the tank design used, a new design was required. The clamping system incorporated the use of one-inch diameter steel studs 2 and 5/8 inch long at 12 inch center to center spacing on the steel frame. Sections of 4[7.25 channel were to be drilled to allow it to slip over the studs. A top four inch by 3/8 inch steel plate was then cut to the same length as the channel and drilled to fit over the studs. The remainder of the clamping system consisted of one inch diameter steel rod, 1 1/4 inch diameter black steel pipe, and a top wood filler. Figure 21 shows a rendering of the clamping arrangement. Each nut was torqued to 150 foot-pounds to develop a continuous seal.

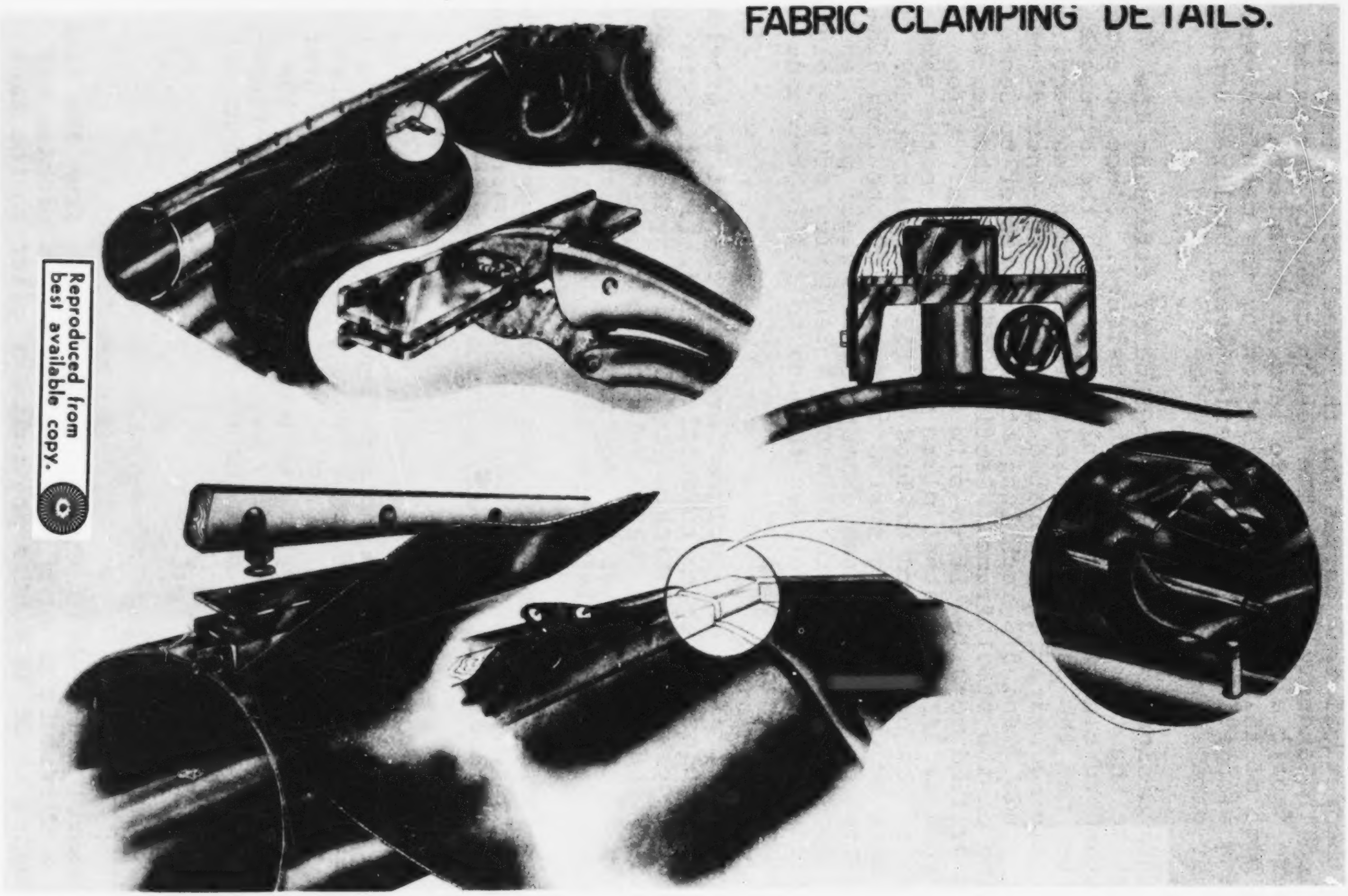
The top steel plate was included over the channel to provide sufficient clamp strength to develop full fabric strength without excessive deflection. The clamping system shown was tested by the Firestone Coated Fabrics Company in California on July 5, 1968. Without the top plate, the clamping system experienced excessive deflection at full material strength.

The excess fabric was wrapped back over the clamping mechanism and fastened down to provide a protective barrier so the tank fabric could not rub against exposed studs and steel plate edges.

The clamping system allows the fabric to take any angle of direction away from the clamp expected to be experienced in tank operation. While the tank frame shape requires that the material in the short sides of the tank be folded over, no excessive wear is expected at these points due to the low internal tank pressures and low tank profile. Figure 21 also shows the folds used for the corner attachment of the tank fabric.

Sedimentation Chamber

The control and removal of undissolved solids from the tank posed a difficult problem. Complicating this problem was the desire to do as little pretreatment prior to the tank


FABRIC CLAMPING DETAILS.

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Fabric Clamping Details

Figure 21

as possible. At the same time, a method had to be developed to allow free discharge from the tank without clogging of the effluent port by the top or bottom fabric during pumping.

During model studies, several locations for influent and effluent ports were investigated. Several protective cages and screens were installed around the effluent port to prevent shutting off of effluent flow during pumping. From these model studies, a method was developed to solve each problem.

A hopper type bottom was developed for the tank. A common influent -effluent port was established in the tank bottom. Over this influent - effluent port a sedimentation chamber was installed.

Figure 22 shows the original sketch of the sedimentation chamber. The major portion of the sedimentation chamber could be constructed from a spun elliptical head commercially available. The designed head was of 1/2 inch steel, 120 inch inside diameter, 30 inches high and had 6 inch straight sides. Theoretical sediment control is:

1. Basic hydraulics states that for incompressible flow the volumetric flow rate, Q, is equal to the average velocity, V, of the incompressible fluid flow times the cross-sectional area, A, of the flow: Q=VA.

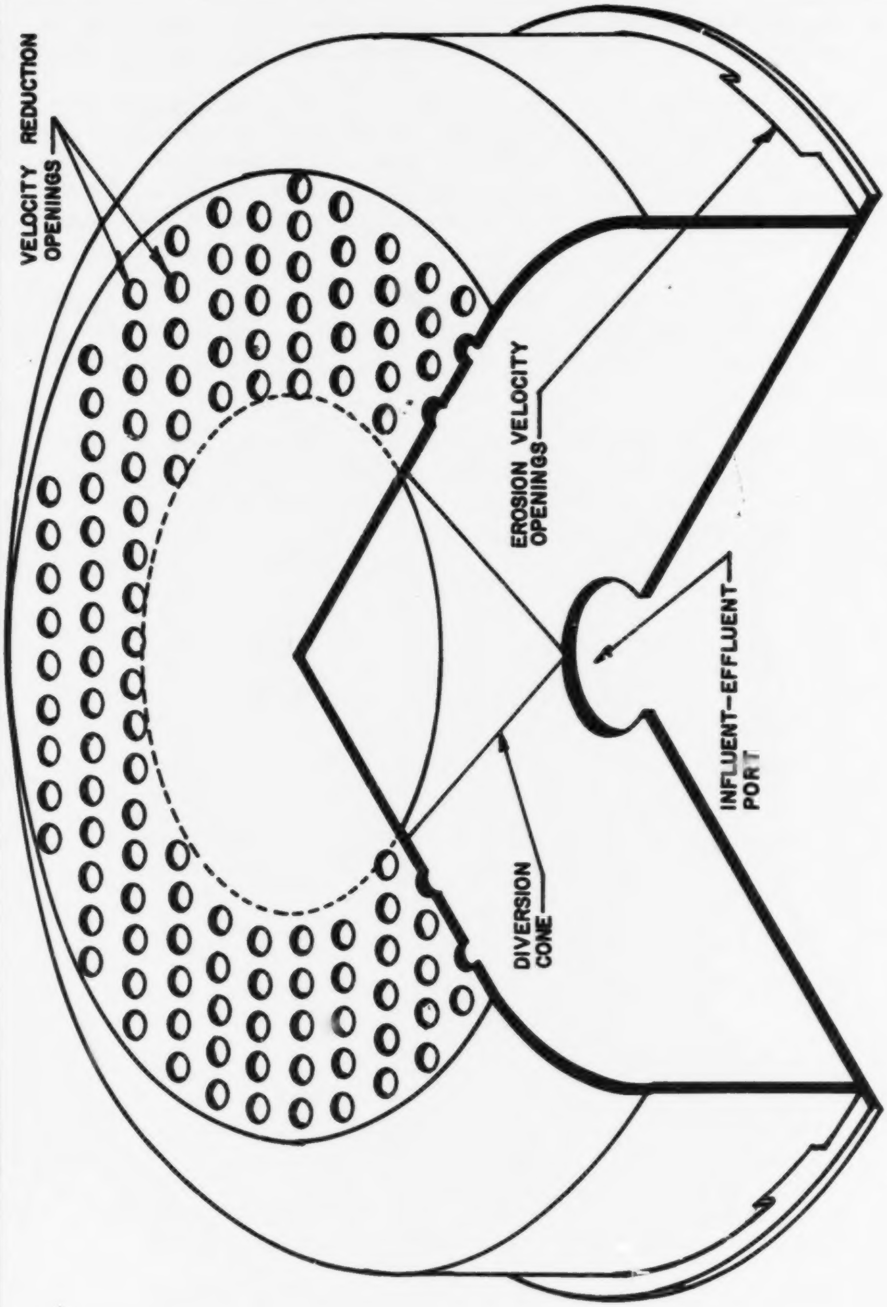
   The continuity equation for incompressible flow states that this flow rate is continuous through a closed system. Therefore:

   $$Q_1 = Q_2$$

   and $$V_1 A_1 = V_2 A_2$$

2. The influent - effluent pipe area is known. A velocity for the influent at the point entering the tank can be estimated for a given flow. If a given open area is provided through the elliptical head over the influent-effluent port, the velocity at the elliptical head surface can be determined.

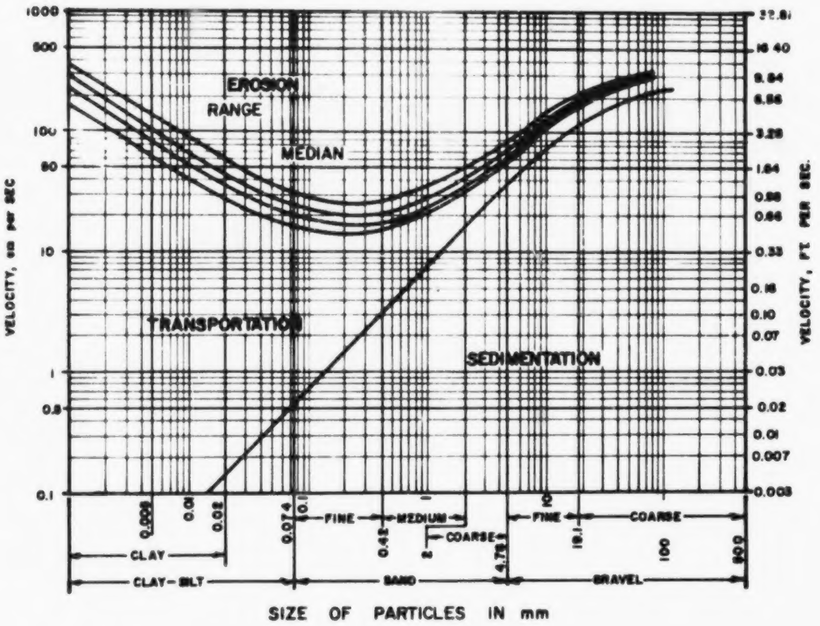
3. Knowing what velocity exists at the elliptical head surface, Figure 23 can be examined to determine what size of particle will begin to settle out. An optimum particle size can


VELOCITY REDUCTION OPENINGS
EROSION VELOCITY OPENINGS
INFLUENT-EFFLUENT PORT
DIVERSION CONE

SEDIMENTATION CHAMBER

KARL R. **ROHRER** ASSOCIATES, INC.
AKRON, OHIO

FIGURE 22



RATIO OF PARTICLE SIZE TO VELOCITY REQUIRED FOR EROSION, TRANSPORTATION, AND DEPOSITION. (AFTER HJULSTRÖM.)

FIGURE 23

be determined and required optimum area of velocity reduction openings can be set. With this system operation, grit and some lighter organic matter can be contained within the sedimentation chamber.

4. A diversion cone was placed over the influent-effluent port inside the elliptical head.

5. For removal of the suspended solids in the chamber, Figure 23 gives a required erosion velocity. The effluent port has a constant area. Therefore, an erosion velocity can be produced by varying cross-sectional area at the sedimentation chamber perimeter; since for a given effluent flow rate, the velocity at the effluent point is known.

6. The area of the erosion velocity openings can be determined and this area removed from the periphery of the elliptical head.

7. The operation of the system is automatic with no moving parts. The top fabric conforms to the tank bottom contours when empty. Influent to the tank raises the tank top on filling. Due to the velocity reduction openings and the resulting increase in flow area, grit and some suspended organic material settle out in the head. Suspended material not remaining in the sedimentation chamber settles out close to the head within the hopper bottom. When the tank is filled, most fine material settles out. On emptying, the solids at the effluent port are removed immediately. The effluent velocities are below the erosion velocities required to remove most of the settled solids. The tank is emptied and the top fabric lowers to the elliptical head. As the tank is emptied, the fabric top closes off the velocity reduction openings. Finally all velocity reduction openings are closed off and the erosion velocity openings increase the flow velocity and resuspends settled material effectively removing the major amount of solids which have

entered the tank. The effective range of this velocity increase is limited due to the virtual mass displacement of liquid which occurs. To help move the solids toward the sedimentation chamber, a flushing system was included in the tank utilizing a high pressure flow of bay water, the pipe frame as a manifold, and drilled openings in the frame as nozzles to push the solids toward the sedimentation chamber.

Any solids left in the tank after pumping should accumulate at such a slow rate as to tend to have a negligible effect. The top flexible material as it lowers tends to limit cross-sectional flow area and move this material toward the sedimentation chamber.

The sedimentation chamber also allows free pumping from the tank. The sedimentation chamber is sized along with the top fabric such that the top fabric can not close off all open areas available for stored liquid to flow to the effluent port until the tank is empty. Some liquid will remain within the tank but this liquid is negligible when compared to tank capacity.

Laboratory testing showed that effective sedimentation control could be accomplished with the sedimentation chamber and only trash removal was required prior to the storage tank.

Gas Vent Valve

The storm overflow to be stored may contain entrained gases which will collect in the tank on filling. Evolved gases may also collect during temporary storage of the liquid due to the volatile organic solids which are present. While gas accumulation rates should be minimal the build up of gas could cause excessive buoyancy forces and excessive fabric stress.

Since no commercially available valve was found which would operate under the environmental conditions involved or produce the desired results, a valve was designed for the underwater storage system. The valve design was tested in the laboratory on a scale model and worked sufficiently well to allow adoption for use.

The gas vent valve required had to operate for three different pressure possibilities:

Case I: Zero pressure differential between tank and external environment, and

Case II: Negative pressure differential between tank and external environment,

Case III: Positive pressure differential between tank and external environment.

In Case I and III, the valve must allow the escape of gas pockets within the tank without allowing the escape of liquid from the tank. These two cases occur in the filling of the tank and after the tank is filled. In Case II, the valve must prevent bay water from entering the tank. This occurs on emptying the tank. Gas expulsion is not considered necessary at this point in tank operation.

The gas vent valve design is based on the buoyancy forces of a submerged ball float versus the gravitational weight of the ball float. The gas vent valve is shown in Figure 24.

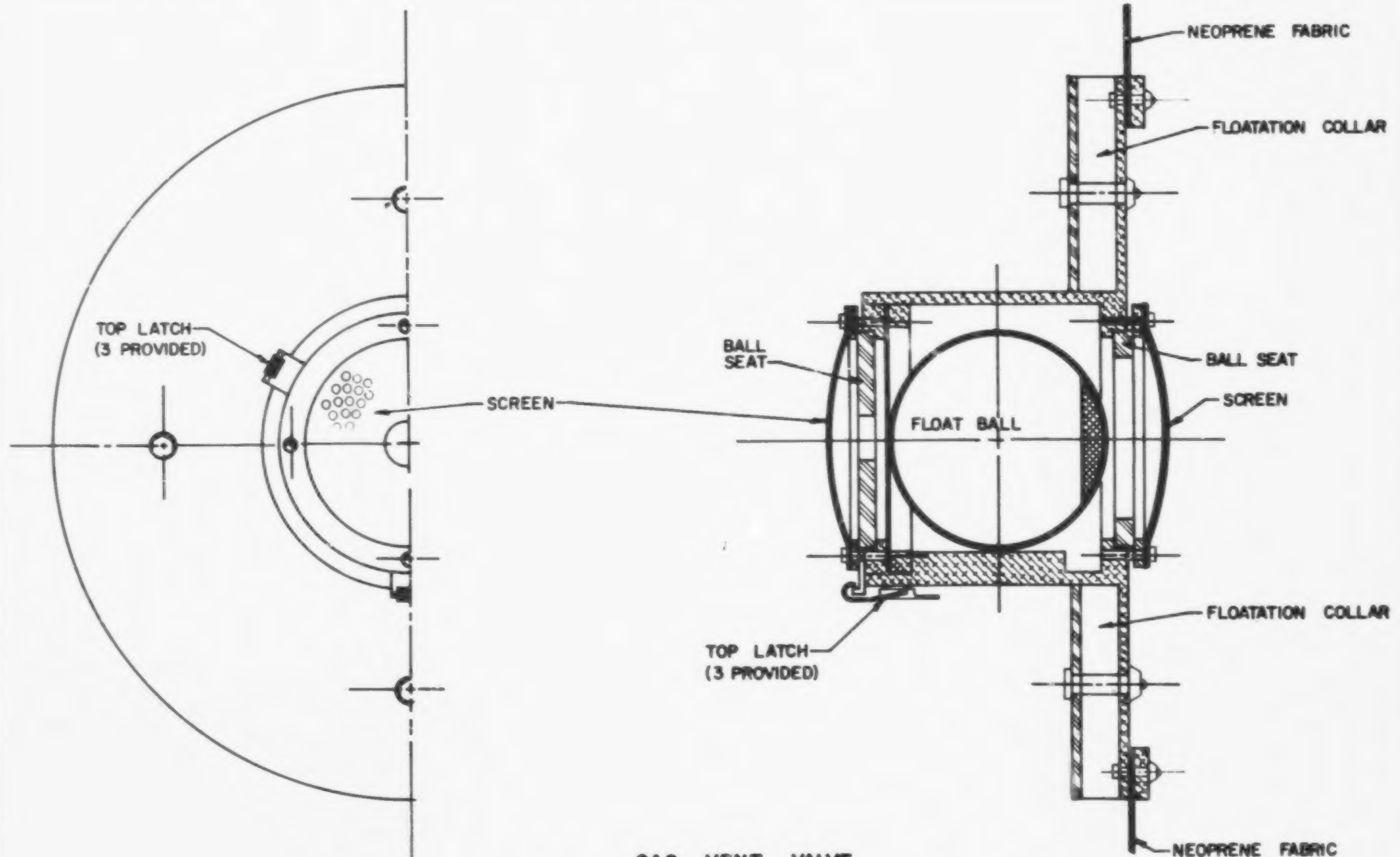
During filling of the tank, there will be zero tank pressure, Case I. The buoyancy of the ball float causes it to seal against the upper seal. When an air pocket forms, the ball float falls free and the gas passes around the ball float until it is again submerged. With an internal pressure, Case III, the buoyancy and internal tank pressure causes the ball float to seal.

When the tank is being emptied, a negative internal pressure is experienced, Case II. The ball float is forced to seat against the bottom seal due to the float weight and pressure differential. Bay water is kept out of the tank by the ball float sealing against the bottom seat.

The density of the ball float and its subsequent submerged weight are critical. Weighted hollow metal spheres were used. Future ball floats could be of plastic materials.

The internal tank pressure must be kept within limits for the gas vent valve to operate as required. However, internal tank pressure was anticipated to be 0.5 psi maximum which allowed full gas vent valve operation.

The ball float tends to operate in a rapid piston type motion. The ball float strength is critical due to this severe loading.


NEOPRENE FABRIC
FLOATATION COLLAR
TOP LATCH (3 PROVIDED)
BALL SEAT
BALL SEAT
SCREEN
SCREEN
FLOAT BALL
TOP LATCH (3 PROVIDED)
FLOATATION COLLAR
NEOPRENE FABRIC

GAS VENT VALVE

KARL R. ROHRER ASSOCIATES, INC.
AKRON, OHIO

FIGURE 24

The gas vent valves were constructed with a float collar to allow a neutral to slightly positive buoyancy. This tends to keep the vent valve at a high point and allows complete expulsion of gas. Five gas vent valves were used on each tank for this installation. On later installations, only one valve should be necessary.

Pressure Relief Valve

With the low tank profile, a pressure relief valve is required for a positive method of preventing internal pressures from exceeding allowable limits. If a low internal tank pressure can be held, fabric strength can be reduced with a substantial reduction in fabric cost resulting.

As a simplification of the fabric tension calculation, the linear tension T (pounds per inch) in the fabric may be said to equal the product of the internal pressure P (pounds per square inch) and the radius of curvature R (inches).

$$T = PR$$

From this, it is evident that the stress in the fabric is directly proportional to the radius of curvature, R, for constant values of internal pressure, P.

Therefore, for a low tank profile with a high radius of curvature, R, a higher stress results than if a higher tank profile was allowed with an associated lower radius of curvature.

The storm overflow to the tank is controlled by a valve in the influent line to the tank. At a preset tank volume or tank height, this valve is automatically closed preventing further storm overflow from entering the tank. Valve failure would result in excess overflow filling the tank and subsequent failure of the fabric might result. To prevent tank failure in case of failure of the control valve, a pressure relief valve was installed just prior to the tank on the influent line. Figure 25 shows the pressure relief valve installed on the west tank.

The top plate was so designed that an internal tank pressure of 0.5 psi would lift the top plate and allow storm overflow in excess of tank capacity to escape to the bay.

After each pressure relief valve was manufactured and the top assembly put together, the assembly was weighed to determine if the submerged weight was within design tolerances.

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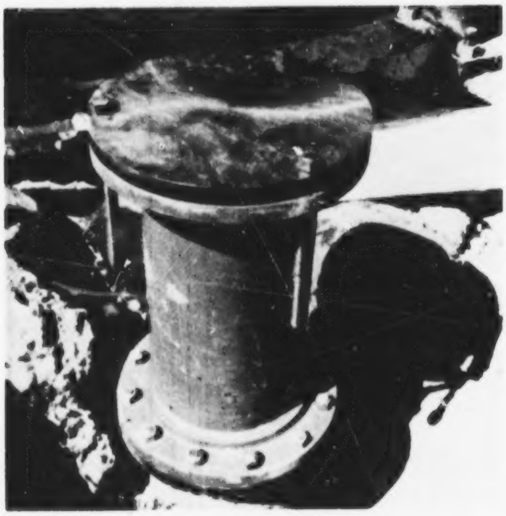


Pressure Relief Valve As
Installed on West Tank

Figure 25

Tank Level Control System (T.L.C.S.)

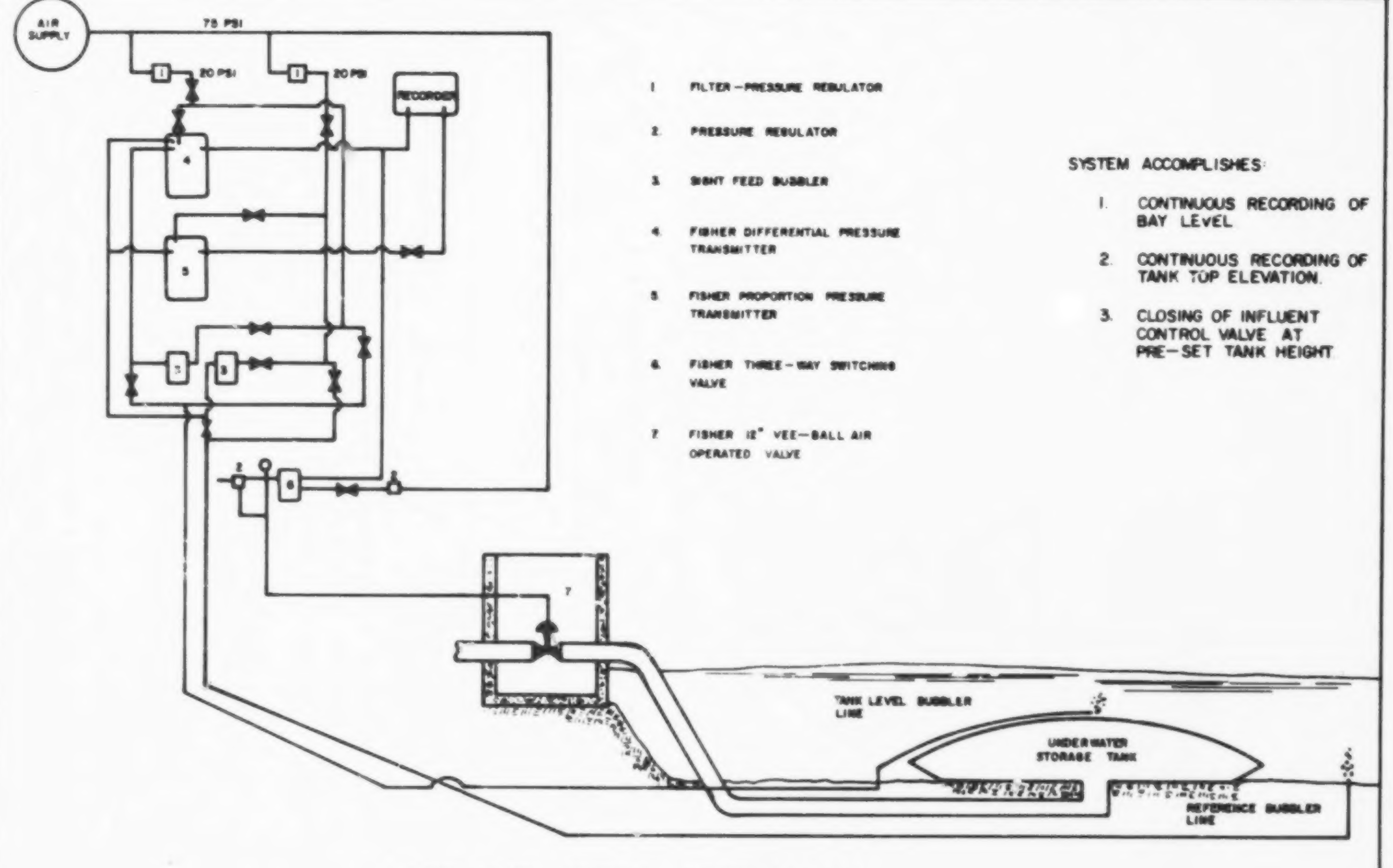
A system was required to continuously monitor tank volume at all times and to operate control valves on the influent lines to the tank. It was also desirable to continuously record bay levels to evaluate bay level effects on the tank system operation.

A bubbler system, Figure 26, was incorporated to yield both tank height and bay level and to operate the tank control valve. With the tank height determined, a plot of tank height versus tank volume can be used to determine tank volume; and, at a given tank height or related volume, the control valves can be signaled to close. Figure 16 shows a plot of tank height versus volume.

Figure 27 shows a photograph of the T.L.C.S. panel in the operator's office as installed during the construction phase of the project.

Connection Chamber

Some structure was required to connect the storage system to the existing outfall. Figure 28 is a drawing of the connection chamber.


AIR SUPPLY
75 PSI
20 PSI
20 PSI
RECORDER
1
2
3
4
5
6
7
1 FILTER—PRESSURE REGULATOR
2. PRESSURE REGULATOR
3. SIGHT FEED BUBBLER
4. FISHER DIFFERENTIAL PRESSURE TRANSMITTER
5. FISHER PROPORTION PRESSURE TRANSMITTER
6. FISHER THREE—WAY SWITCHING VALVE
7. FISHER 12" VEE—BALL AIR OPERATED VALVE
TANK LEVEL BUBBLER LINE
UNDERWATER STORAGE TANK
REFERENCE BUBBLER LINE

SYSTEM ACCOMPLISHES:

1. CONTINUOUS RECORDING OF BAY LEVEL
2. CONTINUOUS RECORDING OF TANK TOP ELEVATION.
3. CLOSING OF INFLUENT CONTROL VALVE AT PRE-SET TANK HEIGHT

TANK LEVEL CONTROL SYSTEM (T.L.C.S.)

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FIGURE 26

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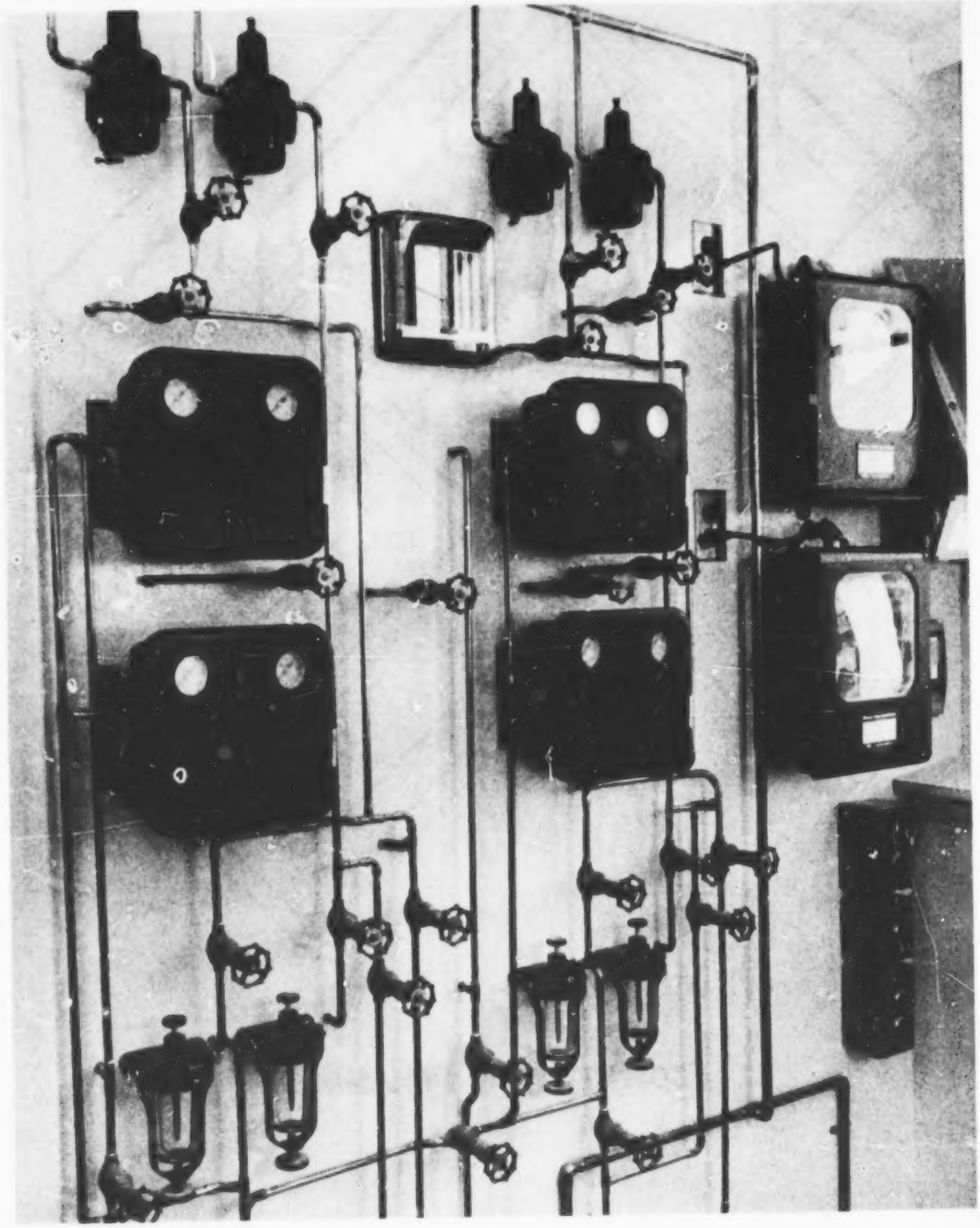
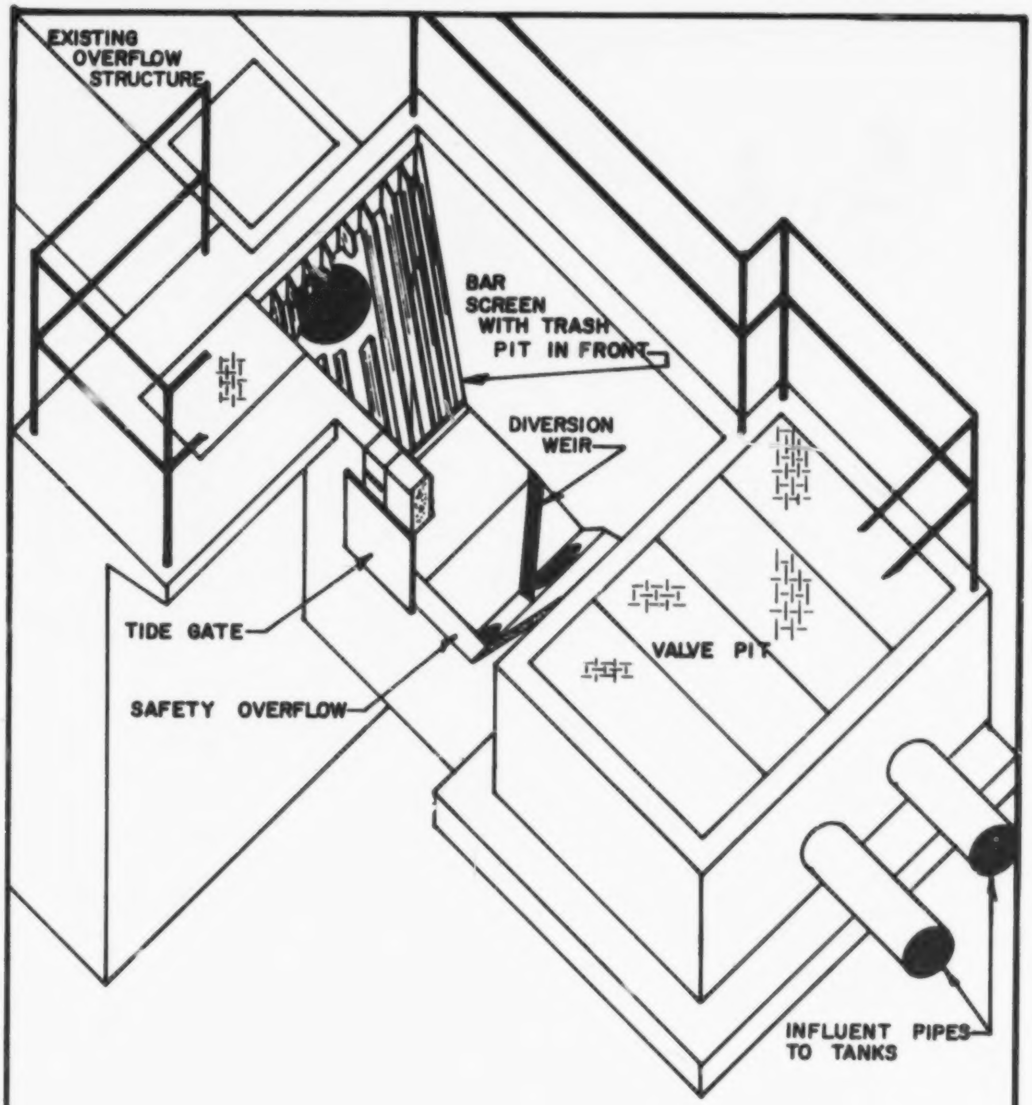


Tank Level Control System (T.L.C.S.) Figure 27


EXISTING OVERFLOW STRUCTURE
BAR SCREEN WITH TRASH PIT IN FRONT
DIVERSION WEIR
TIDE GATE
SAFETY OVERFLOW
VALVE PIT
INFLUENT PIPES TO TANKS

CONNECTION CHAMBER

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AKRON, OHIO

FIGURE 28

The connection chamber is a reinforced concrete structure with two separate compartments. Storm overflow entering the first compartment is first passed through a bar screen to remove all trash from the storm overflow. This trash is caught in a trash pit immediately in front of the bar screen and manually removed after each storm event. Very little trash was anticipated due to the type of drainage area feeding to the McEwen Street outfall. Next, the storm overflow passed to the influent pipes to the tanks. A diversion weir was installed to control storm overflow to one tank or the other. After the tanks were filled, the storm overflow was designed to flow out of the safety overflow provided in the connection chamber for storms in excess of the design storm.

Due to the intensities and durations of storm events in the Northwestern Ohio area, a design capacity to store all storm overflow is infeasible. A contract limitation of 200,000 gallons total tank capacity for the pilot facility also required that some type of overflow be installed.

The flat gradients in the McEwen Street combined sewer system required that minimum restriction of storm overflow be allowed. If storm overflow was backed up when the tanks were filled, or if a higher flow rate of storm overflow occurred than the influent piping to the tanks could handle, the residences in the lower portions of the drainage area would experience flooding of basements. This condition could not be tolerated.

Due to the minimal amounts of trash expected, the safety overflow was placed downstream of the bar screen. On most installations, this safety overflow should be upstream of the bar screen.

The second compartment in the connection chamber housed the influent control valves for the tanks.

Control Building and Associated Equipment

The remaining portions of the storage systems were housed in a control building, a prefabricated metal building 20 feet by 26 feet. The interior of the control building was divided into seven parts to house the remaining portions of the storage system as required by system design. These sections were:

1. Office and instrumentation control room,
2. Pump pit,

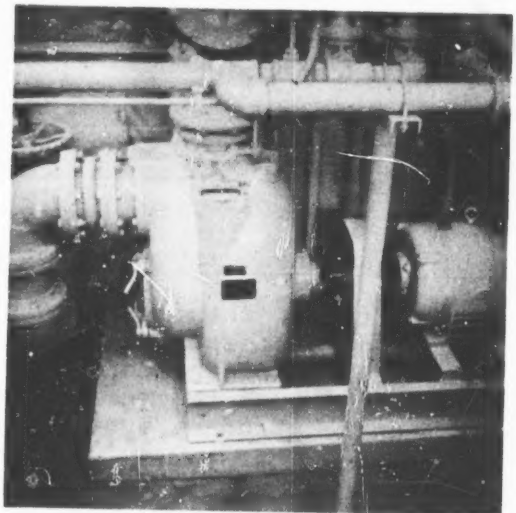
3. Auxiliary power area,
4. Maintenance area,
5. Sample analysis area,
6. Sampler room, and
7. Washroom with shower.

The office housed a desk for the operator and all of the recording equipment. Recorders were necessary for continuous recording of dry-weather sanitry flow, storm overflow, T.L.C.S. showing bay water elevation and tank top elevations, and the volume of liquid pumped from the tanks. The west wall of the office also housed the major portions of the T.L.C.S. instrumentation and all pump controls. See Figure 27.

The northwest portion of the control building was used as a pump pit. An eight-inch centrifugal Gorman-Rupp sewage pump with its associated motor, magnetic flow meter for pump effluent, and piping and valves were housed here. A Worthington high pressure jet pump was also located here with its associated piping and valves to supply lake water to the tank flushing system and to allow monthly testing of the underwater tanks when no storm event occurred. A sampler for taking a composite sample of pump effluent from the tanks was also provided. Figure 29 is a photograph of the sewage pump.

Sewage Pump

Figure 29



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The system design also required an auxiliary electric supply with automatic load transfer capabilities for a power source during storm events in case of failure of the normal power supply. An Onan 15.6 KVA electric generating plant with an automatic load transfer control was included. A propane supply tank was located outside of the control building for fuel supply to the generator.

A work bench with hand tools was installed in the maintenance area of the control building so minor repairs could be made at the site. The southeast corner of the control building housed the sampler room. The air compressors for the T.L. C.S. were also located in the sampler room.

Laboratory space was provided for sample analysis by the operator at the site. A storage refrigerator with BOD incubator, sink and work space, storage cabinets, drying ovens and muffle furnace, and analytical balance were provided with the miscellaneous equipment for the analysis required.

Sampling Program

To be able to design and evaluate an underwater storage system of the type installed, a complete sampling program was set up. For the design of the facility, the chemical and physical properties of both the liquid to be stored and industrial wastes or other chemicals in the water body which would have a deleterious effect on the storage system materials were determined. Deterioration of the system materials must be limited to allow maximum system life, to provide economy, and to minimize system maintenance.

To be able to evaluate the underwater storage system itself, the pollution load intercepted, the pollution load of the liquid after storage, and the reduction in pollution to the receiving water body must be determined. A sampling program was set up to provide data for evaluation of the following:

1. BOD,
2. Coliform,
3. Settleable solids,
4. Suspended solids, SS,
5. Suspended solids volatile, SSV, and
6. pH.

Sample parameters were determined for:

1. Grab samples for the dry-weather flow,
2. Individual timed samples and composite samples of the storm overflow from the combined sewer drainage area,
3. Composite samples of effluent from the storage tanks, and
4. Grab samples of bay water at the outfall in the surrounding bay after each storm event and as required to provide evaluation data.

The sampling of the combined sewer storm overflow was designed to be accomplished by an automatic sampler. The contract required that the sampler begin taking individual samples at the beginning of overflow and take an individual sample at a time interval throughout the storm. A composite sample proportional to flow was to be taken for each storm event.

At the time of design, no sampler was commercially available to do this job and at the same time secure a representative composite sample. Therefore, a sampler was designed and constructed. A schematic of the sampler used is shown in Figure 30. Twenty-four individual sample containers of one pint each and five gallon composite sample container was used. The time interval for the individual samples was set at five minutes but this time interval could be preset for 5 to 60 minutes. The composite sampler was operated from a total flow switch which took a preset sample every 10,000 gallons of flow.

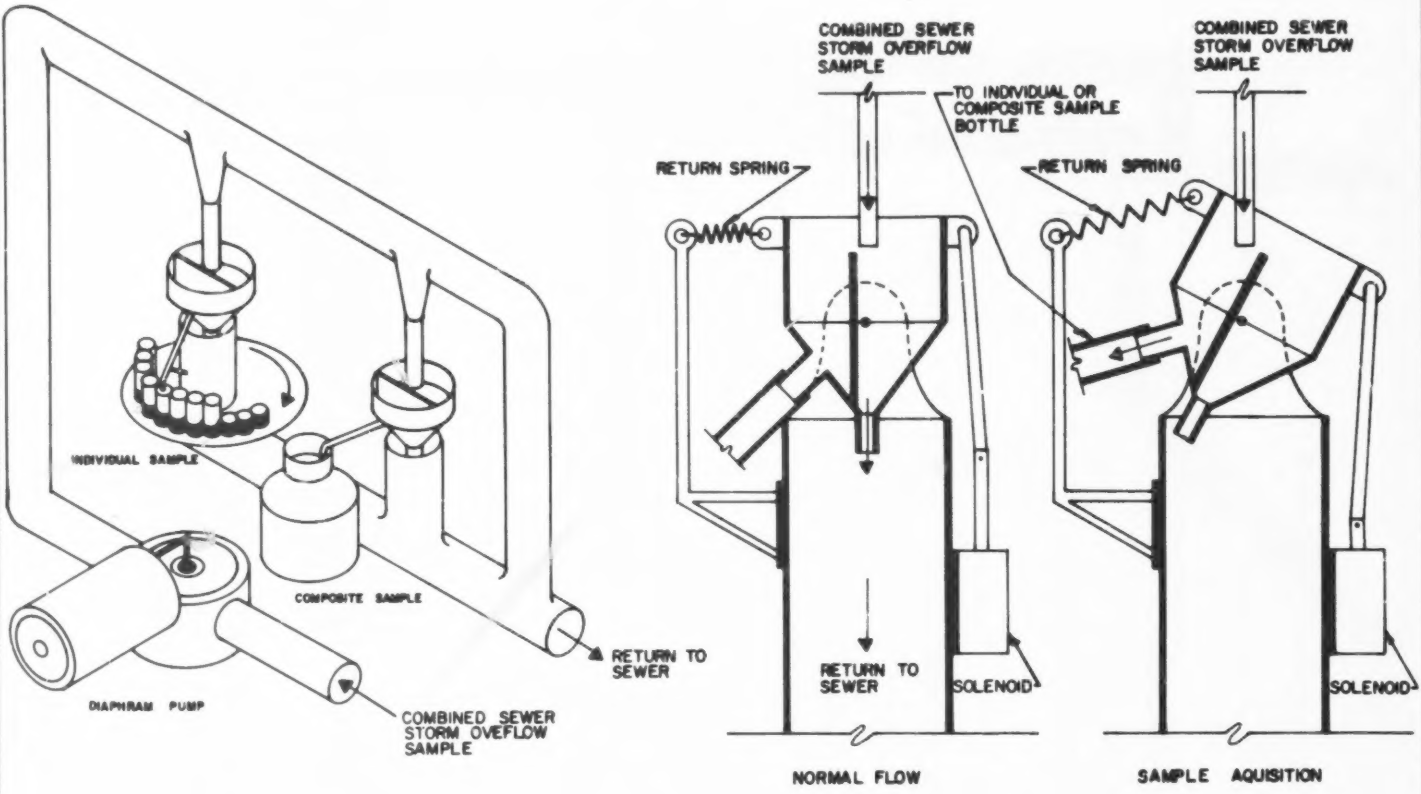
Rain and Staff Gage

In order to evaluate the storage system, a recording rain gage was provided in the preliminary design. Prior to facility construction the rain gage was located in the southeast corner of the drainage area. After construction of the facility, the rain gage was relocated at the site.

A visual method of measuring bay level fluctuation was also desirable to compare with bay level readings from the T.L.C.S. A staff gage was installed outside of the excavated tank area to provide visual readings of water level and peak wave fluctuation which is not recorded by the T.L.C.S.

Design Operation

Figure 31 shows the system flow plan for filling of the underwater tanks during a storm event.


INDIVIDUAL SAMPLE
COMPOSITE SAMPLE
DIAPHRAM PUMP
COMBINED SEWER STORM OVEFLOW SAMPLE
RETURN TO SEWER
COMBINED SEWER STORM OVERFLOW SAMPLE
TO INDIVIDUAL OR COMPOSITE SAMPLE BOTTLE
RETURN SPRING
RETURN TO SEWER
SOLENOID
COMBINED SEWER STORM OVERFLOW SAMPLE
RETURN SPRING
SOLENOID

NORMAL FLOW

SAMPLE AQUISITION

AUTOMATIC SAMPLER

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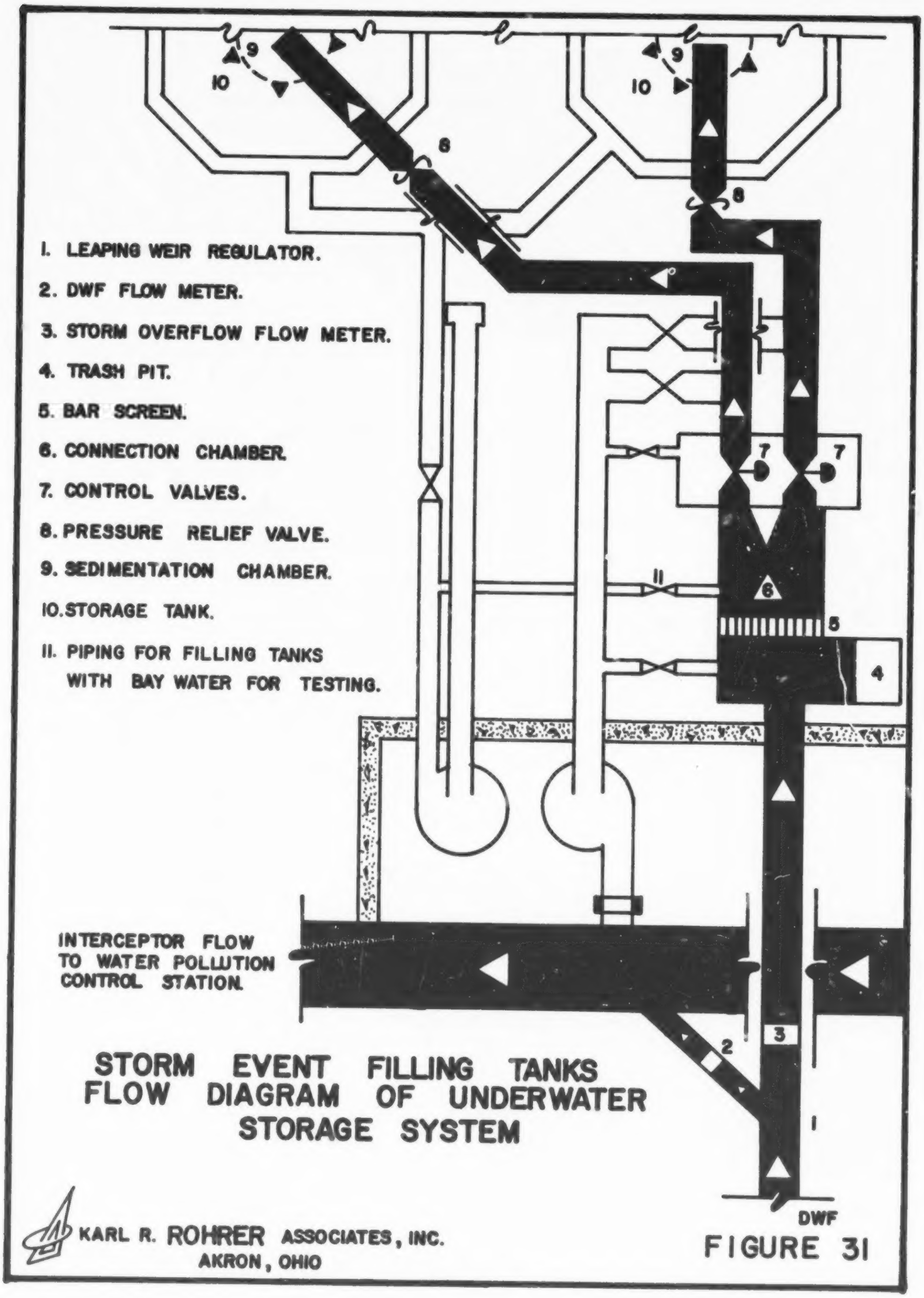
FIGURE 30


1. LEAPING WEIR REGULATOR.
2. DWF FLOW METER.
3. STORM OVERFLOW FLOW METER.
4. TRASH PIT.
5. BAR SCREEN.
6. CONNECTION CHAMBER.
7. CONTROL VALVES.
8. PRESSURE RELIEF VALVE.
9. SEDIMENTATION CHAMBER.
10. STORAGE TANK.
11. PIPING FOR FILLING TANKS WITH BAY WATER FOR TESTING.
9
10
8
9
10
8
7
7
6
5
4
11
INTERCEPTOR FLOW TO WATER POLLUTION CONTROL STATION.
3
2
1
DWF
KARL R. ROHRER ASSOCIATES, INC.
AKRON, OHIO

STORM EVENT FILLING TANKS
FLOW DIAGRAM OF UNDERWATER
STORAGE SYSTEM

FIGURE 31

Runoff from a storm event increases to the point where it jumps the leaping weir regulator at the McEwen Street regulator. Flow not jumping the weir and dry weather flow is intercepted by the weir and is diverted to a second existing chamber where this flow is measured by using a ball float type stage recorder calibrated to an existing channel. After passing through this second chamber, the flow proceeds to the existing interceptor and to the City of Sandusky Water Pollution Control Station for treatment.

Flow leaping the leaping weir passes through an overflow conduit to the connection chamber. Storm overflow is measured just beyond the leaping weir by the use of a ball float in conjunction with a fiberglass flume.

After measurement of the storm overflow, an automatic sampler pumps a continuous liquid sample from the overflow. From this continuous sample flow, individual samples are taken on a preset time interval during the first 120 minutes of the storm event and a composite sample is taken proportional to the flow. The composite sample is taken on signal from the storm overflow flowmeter (once every 10,000 gallons). The operation of the sampler begins on signal of flow by the storm overflow flowmeter.

The main section of the connection chamber houses the trash pit, bar screen, safety overflow, and the influent pipes to the underwater storage tanks. Flow passes through the bar screen with trash falling out into the trash pit. The trash is manually cleaned after each storm event by raking the trash up an inclined ramp to a trash bucket located on top of the connection chamber.

Flow proceeds to the influent pipes to the underwater storage tanks. A diversion weir prior to the influent pipes allows blocking off low flows to one tank of the operators choice so one tank fills leaving the second tank empty unless total storm overflow exceeds the single tank capacity. At higher flow rates, the diversion weir is overtopped, and both tanks filled simultaneously.

The control valves for the influent pipes are installed in the second connection chamber compartment, a valve pit. The control valves remain open until the tanks are filled to a preset volume. On reaching this volume, the T.L.C.S. automatically closes the valves.

After tank capacity is reached and the control valve closes, storm overflow backs up in the connection chamber until depth is sufficient to pass out the safety overflow.

Storm overflow which passed through the influent piping to the underwater storage tanks first reaches the sedimentation control chamber over the influent port of each tank. Most suspended material settles out within this chamber. The storm overflow minus this suspended material continues to the tank proper for storage.

The storm overflow is stored after each storm event until the interceptor sewer has capacity to take the storm overflow to the City of Sandusky Water Pollution Control Station for treatment. Capacity at the Water Pollution Control Station must be available when stored sewage is returned to the interceptor.

Entrained gas and any evolved gases from storage of the storm overflow is removed from the underwater storage tank by automatic gas vent valves located on each tank top.

If failure of the influent control valves occurred, a pressure relief valve located on the influent line to each underwater storage tank allows all flow in excess of maximum design tank capacity to flow to the receiving water body. This is purely a safety feature to prevent storage tank failure.

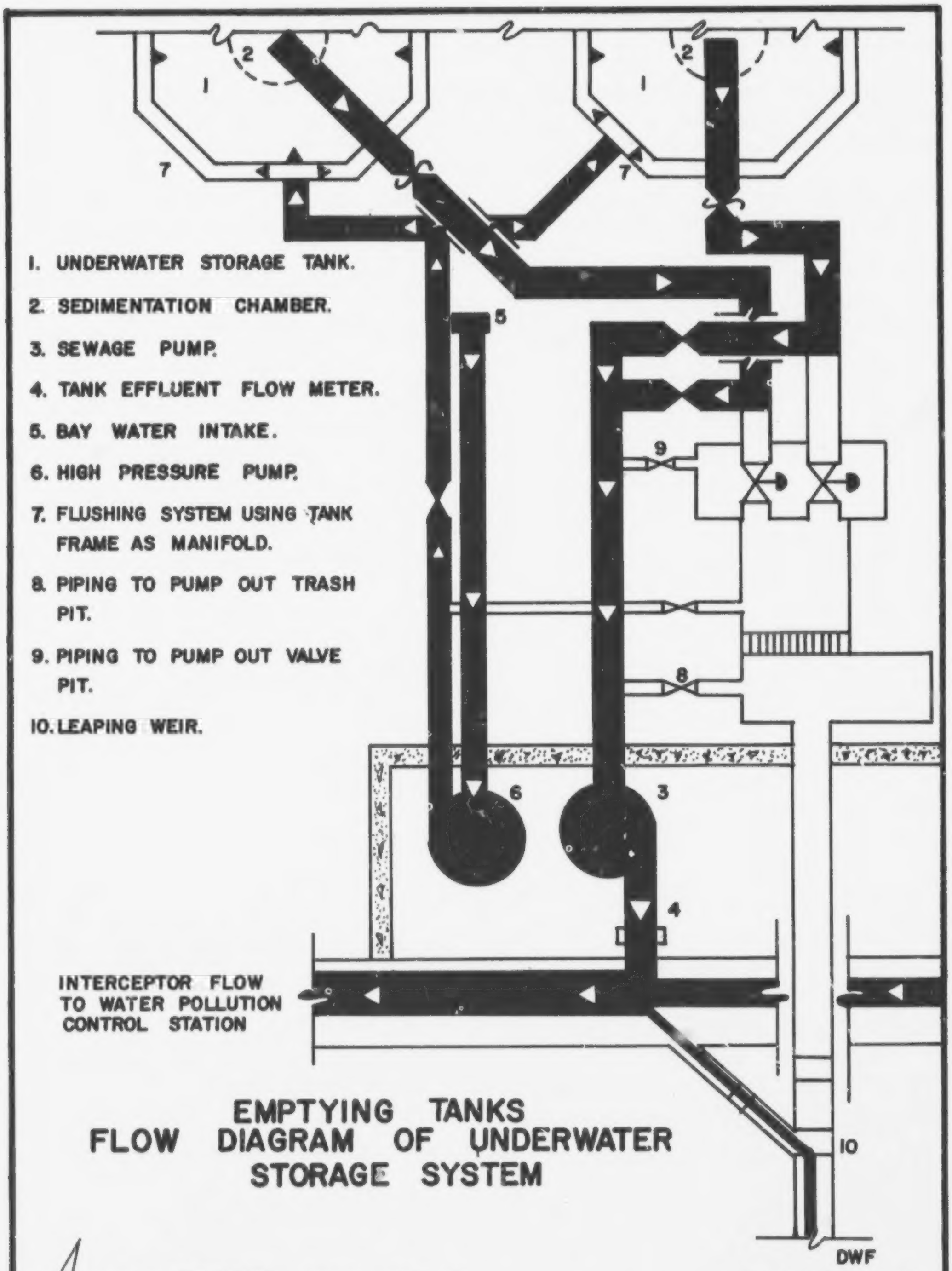
Figure 32 shows the system flow plan for emptying of the underwater storage tanks after a storm event.

Emptying the underwater storage tanks can be accomplished in about one hour per tank. Influent control valves are closed prior to pumping if not already closed from the storm event.

Manually operated valves are opened and the sewage pump manually initiated. One tank is emptied at a time. Effluent from the underwater storage tanks is measured by the use of a six-inch magnetic flowmeter positioned on the effluent piping of the sewage pump.

Grab samples of the storm overflow from the underwater storage tanks are taken on pumping to determine the effects of underwater storage on the pollution load of the storm overflow.

The flushing system is operated in conjunction with the pumping of the underwater storage tanks. A high pressure pump pumps bay water through a water intake in the bay to the tank pipe frame. The pipe frame acting as a manifold directs the high pressure water through nozzles along the


I. UNDERWATER STORAGE TANK.
2. SEDIMENTATION CHAMBER.
3. SEWAGE PUMP.
4. TANK EFFLUENT FLOW METER.
5. BAY WATER INTAKE.
6. HIGH PRESSURE PUMP.
7. FLUSHING SYSTEM USING TANK FRAME AS MANIFOLD.
8. PIPING TO PUMP OUT TRASH PIT.
9. PIPING TO PUMP OUT VALVE PIT.
10. LEAPING WEIR.
1
2
7
INTERCEPTOR FLOW TO WATER POLLUTION CONTROL STATION
DWF

EMPTYING TANKS
FLOW DIAGRAM OF UNDERWATER STORAGE SYSTEM

KARL R. ROHRER ASSOCIATES, INC.
AKRON, OHIO

FIGURE 32

tank bottom. This action tends to flush settled material towards the center of the tank.

After each tank is emptied, the trash in the connection chamber trash pit is manually raked out by the operator. Trash is disposed of at a sanitary land fill.

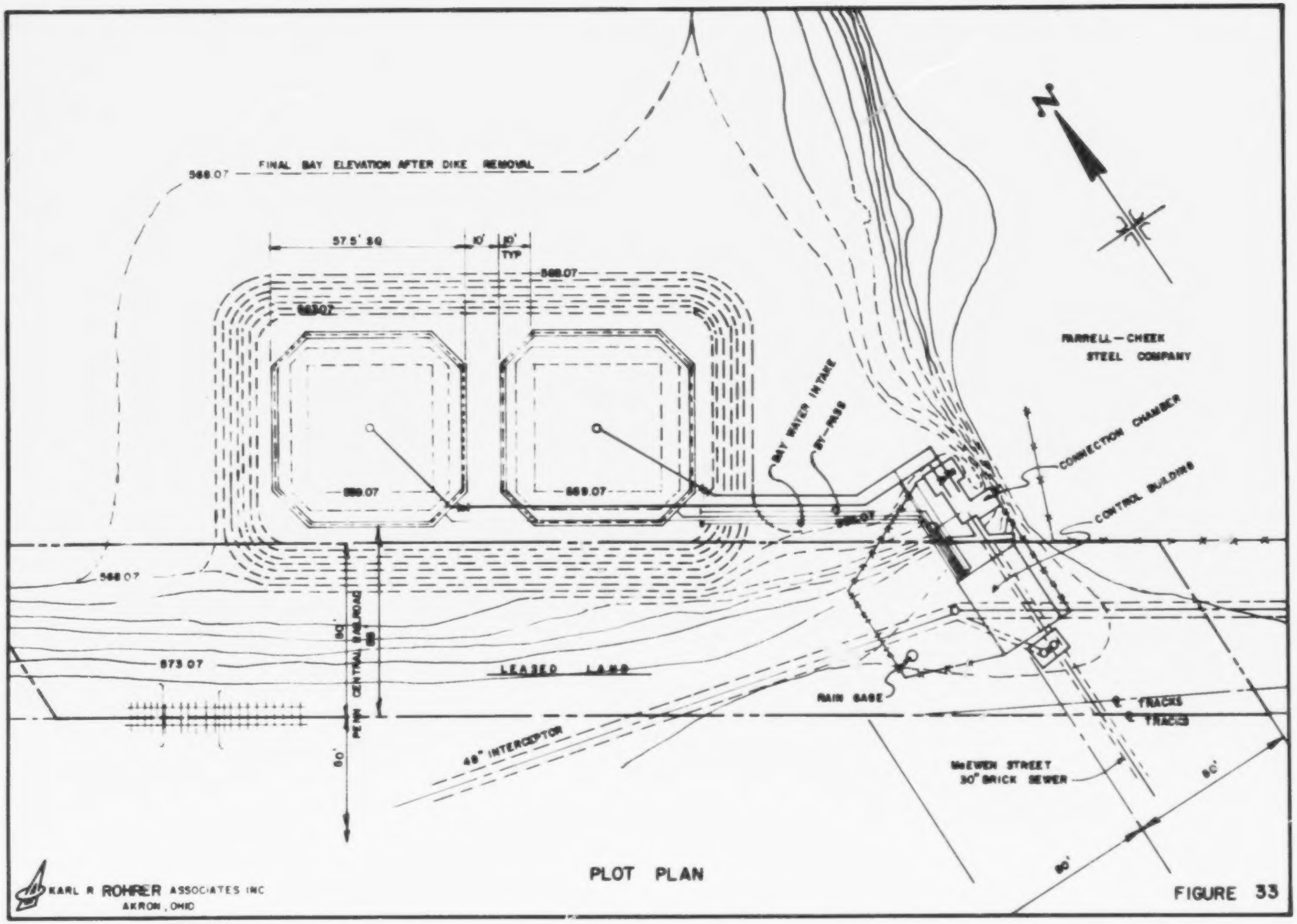
The water in the trash pit is pumped out through a pipe connecting to the sewage pump system.

The connection chamber is hosed out as required. The connection chamber valve pit is pumped out through a pipe connecting to the sewage pump system as required.

During periods when no storm event occurs of sufficient intensity or duration to produce storm overflow, the underwater storage tanks are tested by filling with bay water. Bay water is pumped from a connecting pipe from the high pressure pumping system to the connection chamber. Bay water builds up in the connection chamber until sufficient hydraulic head is available to fill the tank being tested. Only one tank is to be tested at a time to provide capacity in the other tank to intercept any storm overflow from a storm event.

The remainder of filling and testing of the underwater storage tanks is conducted in the same manner as for storm events.

Figure 33 is a plot plan of the underwater storage facility.


568.07 FINAL BAY ELEVATION AFTER DIKE REMOVAL
57.5' SQ
10'
10' TYP
568.07
568.07
569.07
569.07
BAY WATER INTAKE
BY-PASS
568.07
N
FARRELL-CHEEK STEEL COMPANY
CONNECTION CHAMBER
CONTROL BUILDING
568.07
573.07
LEASED LAND
PENN CENTRAL RAILROAD
80'
60'
RAIN GAGE
℄ TRACKS
℄ TRACKS
48" INTERCEPTOR
McEWEN STREET 30" BRICK SEWER
80'
80'
KARL R ROHRER ASSOCIATES INC
AKRON, OHIO

PLOT PLAN

FIGURE 33

SECTION VI

CONSTRUCTION

Construction procedures for the demonstration facility were controlled by the site conditions encountered and by the system itself. Due to the shallow water conditions in Sandusky Bay at the foot of McEwen Street and the elevation of bedrock in the site area, the underwater tanks were located adjacent to the Penn Central Railroad tracks parallel to the waterfront. This tank location required excavation into the bedrock to obtain sufficient depth for the underwater tanks. Tank installation was accomplished by diking in the tank area and pumping the water out for dry construction.

On July 3, 1968, tentative approval for construction was given by the F.W.P.C.A.; and, on July 10, 1968, Rohrer received final authorization to begin construction of the demonstration facility. The subcontract with Bay Construction, Inc. of Sandusky, Ohio was signed and preliminary work already begun was stepped up.

A breakdown of the actual construction time for the demonstration facility is included in Figure 34. The breakdown shows only that time spent for construction and installation on the site and does not include time spent by subcontractors for fabrication of system components. All items marked for a given work day were worked on for at least four hours; however, the entire work day was not necessarily spent on each item. The number of construction workers used is not indicated.

Site preparation consisted of preparing the material and equipment storage area on Farrell-Cheek Steel Company property immediately to the east of the site area. Two temporary crossings were installed over the Penn Central Company tracks for construction. The dike was surveyed and staked so construction could begin.

Construction time was estimated at 100 days with the tank area remaining diked-in until after the underwater portion of the storage system was accepted by the F.W.P.C.A. During the construction portion and while the dike was installed, an overflow pipe from the existing combined sewer outfall beyond the diked area had to be built. This prevented storm overflow from flooding the diked area during construction and also prevented storm overflow in excess of tank capacity from filling the diked tank area while the dike remained in place.

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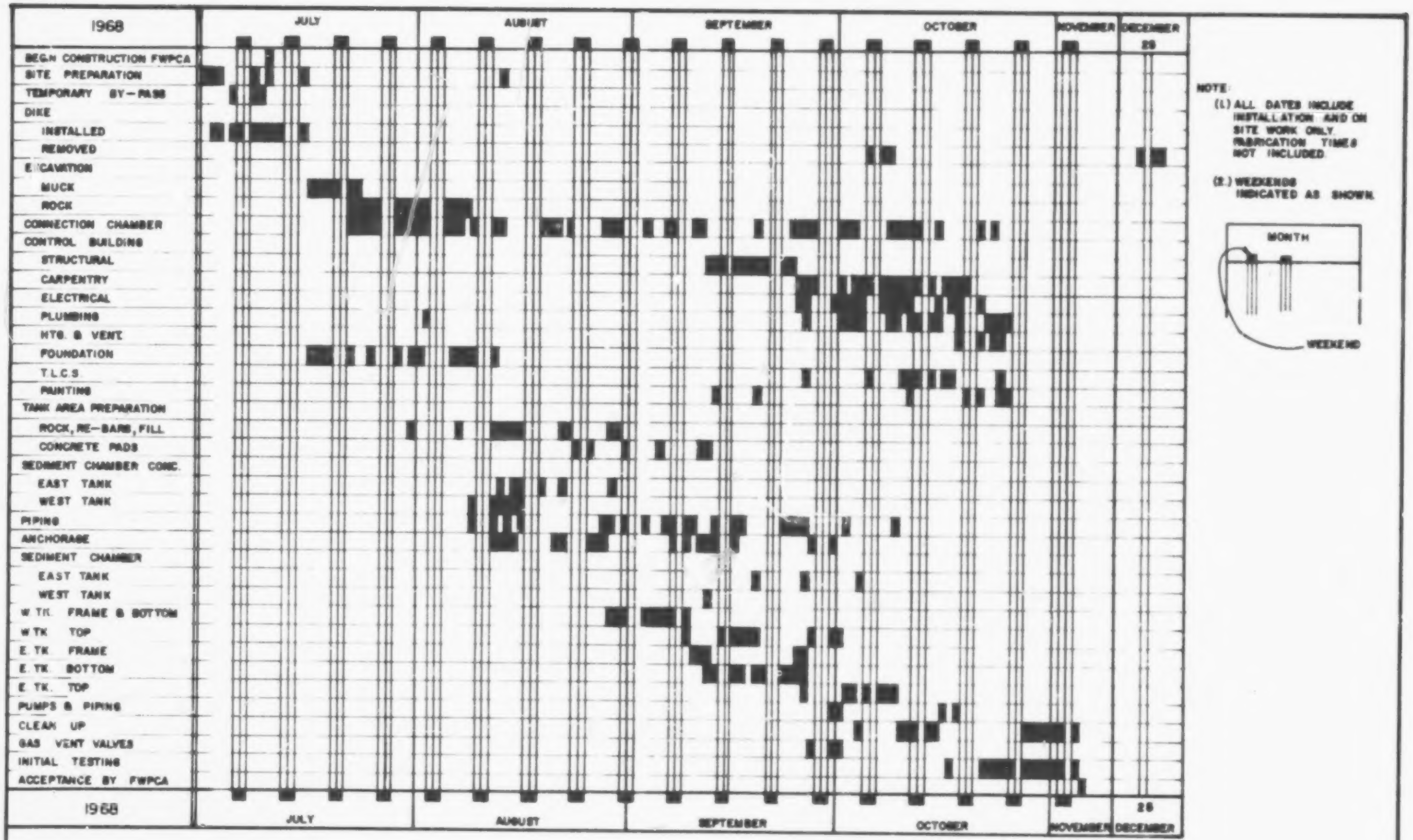


NOTE:
(1.) ALL DATES INCLUDE INSTALLATION AND ON SITE WORK ONLY. FABRICATION TIMES NOT INCLUDED.
(2.) WEEKENDS INDICATED AS SHOWN.

CONSTRUCTION

KARL R. ROHRER ASSOCIATES, INC.
AKRON, OHIO

FIGURE 34

The overflow pipe consisted of a 30-inch tar coated corrugated steel pipe approximately 120 feet long. A tide gate was installed on the bayward end to prevent bay water from flowing into the diked area.

With the tanks located near the existing outfall, an L-shaped dike, approximately 350 feet long, was required. The dike was constructed with sandstone and sealed with clay.

Excavation for the installation of the storage tanks was required due to the existing shallow water conditions. Total design tank height was seven feet for a normally filled tank. Because three to four feet of water was needed over the tanks for best protection, a total of ten to eleven feet of total depth was required at the tank area. To get this depth, it was necessary to remove sediment deposits over existing bedrock and remove or excavate some bedrock to achieve a bottom tank elevation of 559.07.

Overlaying deposits of silt, clay, and sand with some loose rock were removed from the diked area after dewatering. A total of 3,080 cubic yards was removed and placed on upland property above high water level as required by the F.W.P.C.A. Great Lakes Office. This top material removed had a high organic content due to the combined sewer outfalls along the bay and the municipal treatment facilities which use the Sandusky Bay and Sandusky River as receiving water bodies for effluent.

Since bedrock in the vicinity of the tank area did not allow installation without blasting, rock was excavated by blasting to a depth which allowed placement of feed pipes to the tanks and then gravel backfill was used to establish required final elevations.

Blasting of the rock was accomplished by drilling with a rotary air drill, then placing blasting mats over the charges in the drilled holes and over the rock to be removed. This was required since homes were located 70 yards from the tank area. Rock excavation depth varied from 3.8 to 5.6 feet over the entire tank area resulting in a total of 2,205 cubic yards of rock excavation.

Tank Installation

After all material was excavated, the diked area was ready to be prepared for installation of the two tanks. Each tank required eight anchor piles. Holes were drilled into the bedrock and four anchor bolts were grouted into place for each pile. The anchor pile base plates were then slipped over the anchor bolts, leveled at the proper elevation, bolted down, and grouted into place.

On August 7, 1968, anchor bolts were tested to determine if sufficient strength could be developed. Loads were applied up to twice the design loads with no sign of failure.

The next step in tank design was the construction of the center concrete support for the sediment control chamber. Upon completion of the center concrete support, the tank area was brought up to the required elevations with gravel. A concrete pad was then poured to fit the bottom contours of both storage tanks. The concrete pad for the east tank was approximately six inches below final tank elevation, since a sand cushion was placed between the pad and the rubber coated fabric bottom of the east tank to reduce abrasion on the fabric.

The use of the concrete pad allows uniform support for the tank bottoms. The support is necessary due to possible sediment accumulation in the tank and underwater forces exerted in the tanks. Future installations might not allow dry construction; however, storage tank design could easily be altered to accomodate wet installation.

The steel components were manufactured by Continental-Fremont, Inc., in Fremont, Ohio approximately 26 miles from the Sandusky, Ohio site. The east tank frame was manufactured in four quarter sections for shipping and installation. The west tank frame was manufactured in four quarter sections with the 1/4 inch steel bottom plate attached. The remaining steel plate for the bottom of the west tank was then delivered in six sections.

One 18-ton and one 25-ton truck mounted crane with telescopic boom were used to lower each quarter section into its proper location for attachment to the anchor piles. Each quarter section for the west tank weighed about 9,700 pounds. Each quarter section for the east tank weighed about 5,650 pounds.

West Tank

With concrete pad, influent piping, center concrete support structure, and anchor piles completed, the quarter sections of the tank frame were lowered into their respective approximate locations. Final placement was completed and the quarter section was welded to two anchor piles. In turn, each quarter section was then welded together.

The six bottom sheets were then positioned and welded together. The sediment control chamber and its bottom sheet were then lowered and bolted into position over the center concrete structure. Bottom support was supplied by the concrete pad. See Figure 35 and Figure 36.

The 18-inch steel pipe tank frame was tested to insure water tightness by filling with water for four hours at 125 psi. Holes, three sixteenth inch in diameter were then drilled in the frame as called for in the design of the flushing system to aid in sedimentation removal. The bottom of the tank was then filled with water to check the water tightness of the west tank bottom.

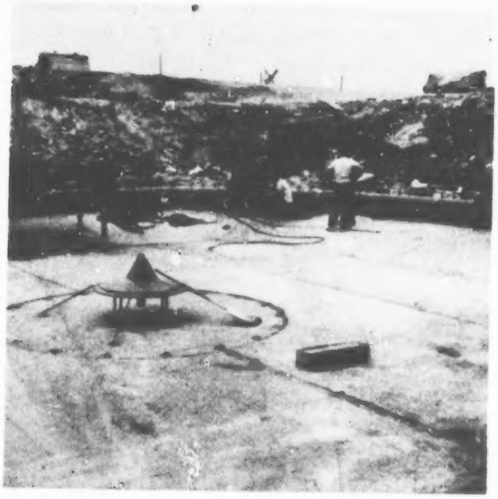
The top fabric was installed next. Mr. M.M. Yancey, Technical Representative for the Firestone Coated Fabrics Company, supervised installation of the rubber coated fabric purchased for the underwater tanks.

For the west tank, the fabric was unrolled in the tank bottom. A minimum of seven laborers was required. The tank was then filled with water to reduce the dead weight of the fabric for installation. This allowed a reduction in fabric elongation during installation and a more accurate placing of the fabric in the clamping system. The weight of each fabric section was about 3,550 pounds.

Care must be exercised in securing an exact fabric length between tank sides. By being only two to four inches off on the fabric clamp line in the 56.58 foot clamp to clamp fabric design length, a variance in excess of 10,000 gallons of tank volume could result at zero psi internal pressure. Mark lines for clamping the fabric had been put on by the manufacturer.

After clamping the fabric to the frame, the excess fabric was pulled back over the clamp and fastened to the side of the clamp channel. Figure 21 in Section V, is a rendering of the clamping system used.

Five gas vent valves were installed in the top tank fabric. A bubbler line was attached along the tank fabric from the frame to the center gas vent valve. The exposed tank metal was painted with an aluminum base paint.



West Tank
Diversion Cone

Figure 35

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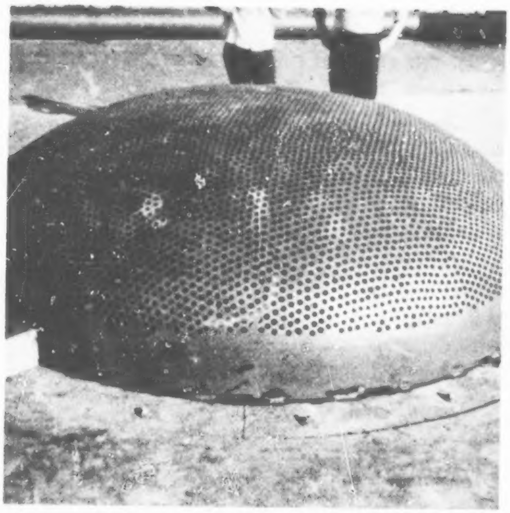
West Tank
Sedimentation Chamber

Figure 36



East Tank

Preliminary work was completed for the east tank in a manner similar to the west tank. The east tank had a rubber fabric top and bottom which presented some difficulty in installation. The anchor piles were constructed four feet longer than necessary for final tank elevation. The four tank frame sections were lowered into their relative positions for fabric installation, welded together, and bolted to the anchor piles four feet above final tank elevations. A wooden platform was installed beneath the east tank frame and the bottom tank fabric unrolled on this platform. The fabric was then clamped to the frame and the wooden platform removed. The frame was then unbolted from the anchor pedestals and lowered to its final position.

Both sedimentation chambers for the tanks arrived two weeks late. Therefore, the bottom fabric for the east tank was installed prior to the installation of the sedimentation chamber. An opening was cut in the bottom fabric to allow the sedimentation chamber to be positioned as required. The sedimentation chamber along with its bottom sheet was welded to the bottom steel plate over the center concrete structure.

The top fabric was then lowered into the tank and unrolled. Clamping of the fabric was completed; and, after installation of the five vent valves, bubbler tube, and painting of exposed metal, the east tank was ready for preliminary testing.

When the east tank was filled with water the first time, a leak developed where the bottom fabric was fastened by the bolt ring to the center concrete structure. The influent-effluent pipe to the tank had been installed in a 18-inch wide trench through the concrete pad and the space around this pipe packed with sand. Prior to testing, the sand had settled at the edge of the center concrete structure and the fabric failed at the bolt ring due to the weight of the water over it, and sand washing out of the pipe trench.

To repair the leak, the top fabric was unclamped along one side and then the bolt ring holding the bottom fabric removed. The sand in the influent - effluent pipe trench to the center concrete structure was removed from around the pipe and replaced with grout. A steel plate was clamped to the bottom fabric and the bolt ring tightened. The top fabric was reclamped and the tank refilled with water.

No leaks reoccurred. Figure 37 shows the two completed underwater storage tanks in October, 1968.

Control Building

A 20 foot by 26 foot prefabricated metal building to house the system pump, pump controls, bubbler system, and recording instrumentation, sampler, and auxiliary generator arrived at the site eight weeks after it was scheduled. This prevented system completion by the October 16, 1968 end of construction date anticipated. However, after arrival, the control building and associated equipment was installed in time for the rescheduled October 26, 1968 opening.

All system piping was installed as required. Twelve-inch lines to the tanks were installed in trenches blasted in the rock. A water intake was placed in the excavated tank area to provide all water necessary for testing the storage facility and to supply bay water for the high pressure flushing system.

Construction Complaints

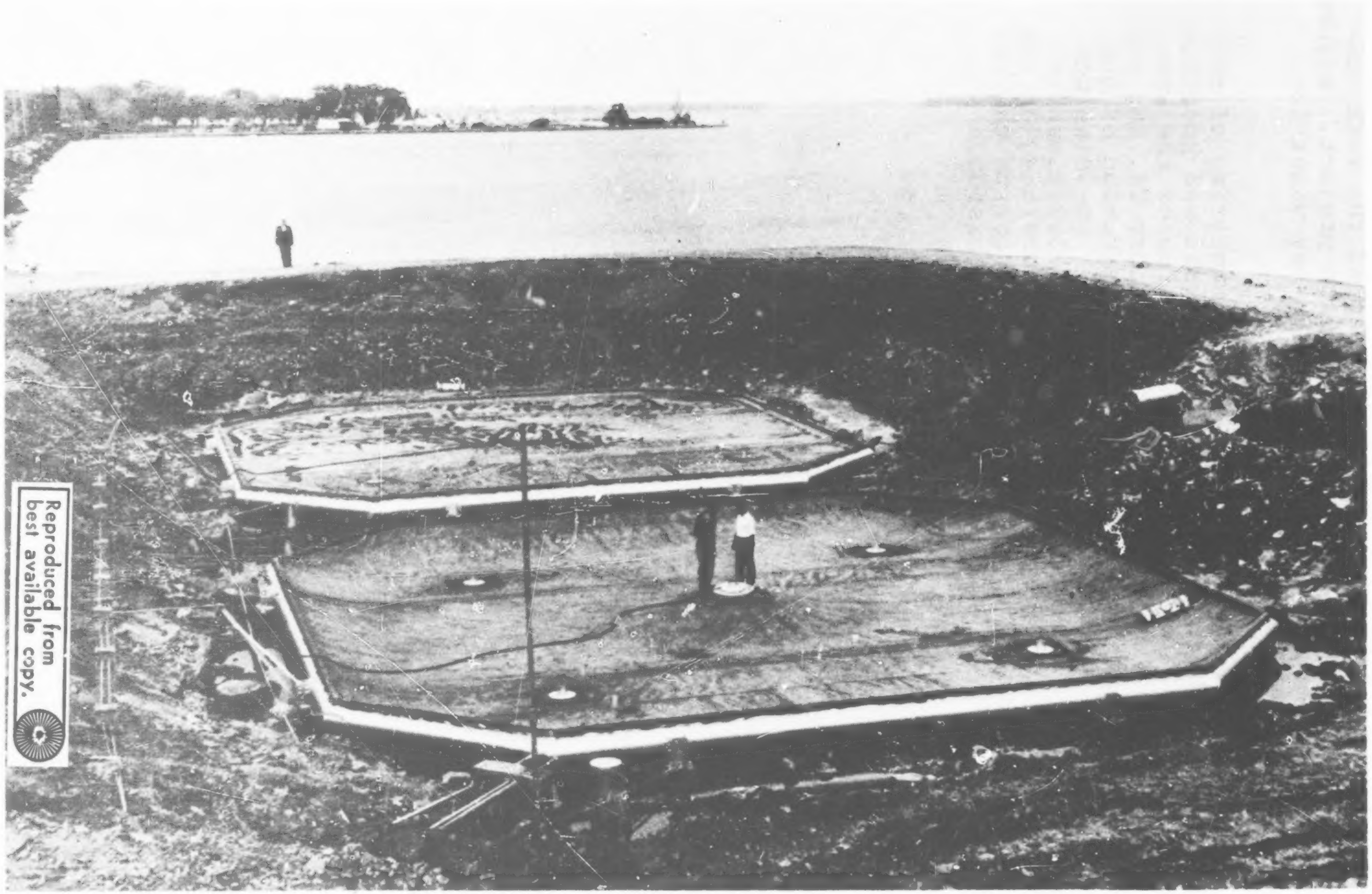
No complaints were registered concerning the noise or blasting which accompanied drilling and rock excavation. However, complaints were heard concerning the dirt which was deposited on Ogontz Street during dike installation, muck excavation, and rock removal. Also complaints concerning the heavy truck traffic were noted.

Preliminary Testing

After completion of construction, preliminary testing was conducted from October 17, 1968 through November 5, 1968 to determine if the underwater storage system was operating properly and to check instrument calibration. Several minor problems were discovered and corrected. These were:

1. Leaking check valve in connection chamber,
2. Unstable concrete pedestals for bubbler lines to underwater tanks,
3. Defective switch on air compressor,
4. Excessive vibration in Gorman-Rupp sewage pump, and
5. Missing operator key for valves.

The construction overflow pipe was sealed at its junction with the outfall structure.



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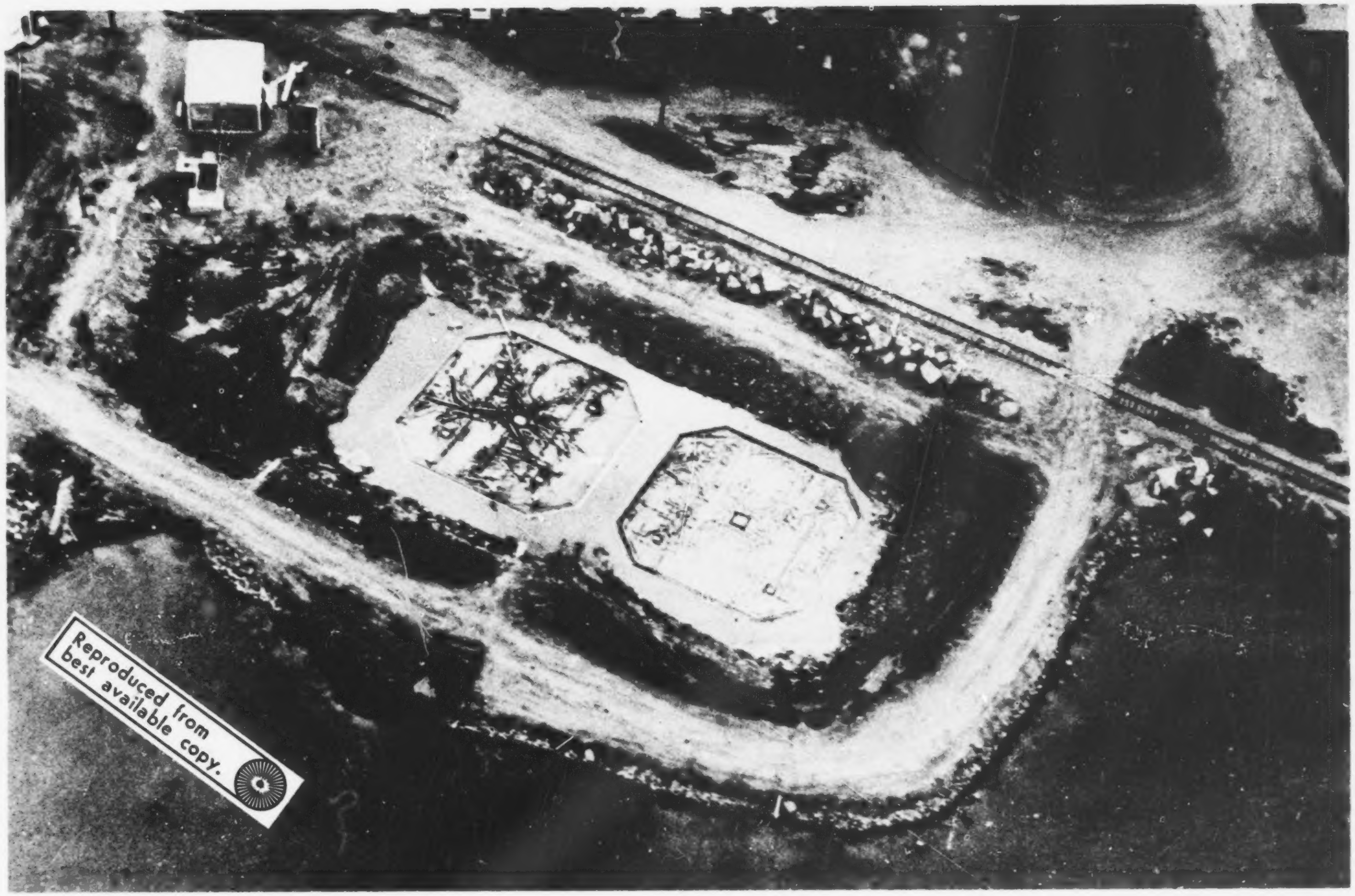
UNDERWATER STORAGE TANKS

Figure 37

On November 5, 1968 the F.W.P.C.A. accepted the work completed under the construction phase of the project. Figure 38 shows the dike area being filled with bay water for preliminary testing.

The dike remained in position to allow dewatering of the tank area until December 24, 1968. This allowed possible repairs to be made on the underwater portion of the storage system. None were required. On December 24, 26, and 27, 1968, the dike was removed to allow operation of the system in actual environmental conditions. The dike was removed to a depth below the original bay elevations to provide a place for the silt which had accumulated on the outside perimeter of the dike to deposit prior to the excavated tank area. This was partially successful.



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UNDERWATER STORAGE TANKS Figure 38

SECTION VII

OPERATION PERIOD

The one-year operation period for the underwater storage system was conducted from November 5, 1968 to November 5, 1969. During this period, the basic operation of the underwater storage tank was shown to be successful. However, mechanical difficulties occurred due to the numerous new systems involved and new adaptions of equipment. Mechanical failure also occurred in some of the standard monitoring equipment. Record high lake levels and its associated problems along with mechanical difficulties of monitoring equipment prevented acquisition of all data required for system evaluation.

The following pages contain a discussion of the testing and storm events which occurred during the one-year period. Detailed explanation of operation difficulties are given as necessary.

Table 8 lists the periods for which the storage tanks were in operation. The dates of the event, the volume stored, the lake level, and pertinent operational comments are included. The variance in times between filling of the tanks and emptying of the tanks is due to the operator's schedule and in certain cases to planned activities concerning one or both tanks. With the sewage pump used, 100,000 gallons of liquid could be pumped from the tanks to the interceptor in about 60 minutes. However, using the high pressure pump to fill the tanks during tests required from eight to twelve hours each, depending on bay level.

During the year of operation, a total of 4,825,000 gallons of liquid was stored in the two underwater storage tanks of the pilot facility. Table 10 shows a breakdown of the volumes for storm overflow and testing and the number of times each tank was in operation. A total of 988,000 gallons of storm overflow was intercepted and returned for treatment by the pilot facility.

With continuous use of the monitoring equipment and operator's observations of instrumentation, operation problems which had not shown up in the initial testing were encountered. Certain of these involved difficulties which persisted throughout the year;others only required slight system modification or adjustment for elimination.

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TABLE 8
TANK STORAGE DATA

*(F) Date Filled
(E) Date Emptied

*Date	Filling		Volume Stored (Gal)		- Lake Level -	Comments
	Test	Storm	East	West		
1968						
Oct.					Monthly Max. 570.0	
25 (F) 26 (E)	X		102,960		570.0	Preliminary Testing
25 (F) 26 (E)	X			116,000	570.0	Preliminary Testing
- BEGINNING OF OPERATION PERIOD NOVEMBER 5, 1968 -						
Nov.						
6 (F) 14 (E)	X		99,900		570.0	
6 (F) 6 (E)	X			98,000	571.4	
Dec.						
26 (F) 26 (E)			101,750		569.4	

TABLE 8
TANK STORAGE DATA

Date	Filling		Volume Stored (Gal)		- Lake Level -	Comments
	Test	Storm	East	West		
Dec.						
28 (F) 29 (E)		X	104,340		570.0	Storm Intensity 0.5 In./Hr. Max. 0.116 In./Hr. Average
28 (F) 29 (E)		X		91,200	570.0	Storm Intensity 0.5 In./Hr. Max. 0.116 In/Hr. Average
1969						
Jan.					Monthly Max. 570.4	
22 (F) 23 (E)	X		190,875		569.1	East Pressure Relief Valve Open
22 (F) 23 (E)	X			82,125	569.1	
Feb.					Monthly Max. 571.9	
15 (F) 16 (E)	X		140,000		571.5	Staff Gauge Removed By Ice

TABLE 8
TANK STORAGE DATA

Date	Filling		Volume Stored (Gal)		- Lake Level -	Comments
	Test	Storm	East	West		
15 (F)						
17 (E)	X			72,000	571.5	
17 (E)	-		30,600		571.2	Leaked In During Night-Pressure Relief Valve Open
26 (F)						
26 (E)	X			100,375	571.2	
27 (F)	X		20,000		571.2	To Check Leak
March					Monthly Max. 572.0	
13 (F)						
13 (E)	X			99,000	570.7	East Tank Not In Operation
31 (F)						
31 (E)	X			108,000	571.3	East Tank Not In Operation
April					Monthly Max. 573.5	
9 (F)						
10 (E)			90,000		573.5	1# Dye -Testing for Leak

TABLE 8
TANK STORAGE DATA

Date	Filling		Volume Stored (Gal)		-Lake Level -	Comments
	Test	Storm	East	West		
14 (F)						
16 (F)						
23 (E)	X		171,000		571.4	5# Dye Testing for Leak
18 (F)						
21 (E)		X		117,000	572.3	Storm Intensity .17 In./Hr. Max. .079 In./Hr. Average
May					Monthly Max. 573.0	
10 (F)						
11 (E)		X	183,000		572.5	Storm Intensity .150 In./Hr. Max. .074 In./Hr. Average
10 (F)						
11 (E)		X		88,600	572.5	Storm Intensity .150 In./Hr. Max. .074 In./Hr. Average
June					Monthly Max. 572.4	

TABLE 8
TANK STORAGE DATA

Date	Filling		Volume Stored (Gal)		- Lake Level -	Comments
	Test	Storm	East	West		
11 (F) 14 (E)	X		Not Available		571.9	
11 (F) 14 (E)	X			Not Available	571.9	Filled For Cleaning Water Removal Not Checked By Operator
15 (F) 23 (F) 26 (E)	X		Not Available		572.1	Filled For Cleaning Water Removal Not Checked By Operator
24 (F) 26 (E)	X			Not Available	572.1	Filled For Cleaning Water Removal Not Checked By Operator
July					Monthly Max. 573.0	
4 (F) 12 (E)		X	109,000		572.2	Elev. of Tank Top 571.1

TABLE 8
TANK STORAGE DATA

Date	Filling		Volume Stored (Gal)		Lake-Level-	Comments
	Test	Storm	East	West		
4 (F) 10 (E)		X		Not Available	572.4	Air Leak Caused Lake Level to Empty Tanks. Recorder to West Tank Not Functioning. 5.8 Inches of Rain during Storm.
17 (F) 20 (E)		X	Not Available		572.2	Elev. of Tank Top 569.9
17 (F) 17 (E)		X		Not Available	572.2	Elev. of Tank Top 570.4 Air Leaks Allowed Lake Level To Empty Tanks.
Aug.					Monthly Max. 572.7	
6 (F) 8 (E)	X		241,500		572.1	Elev. of Tank Top 569.9
5 (F) 8 (E)	X			180,000	572.1	West Tank Recorder Not Functioning

TABLE 8
TANK STORAGE DATA

Date	Filling		Volume Stored (Gal)		- Lake Level -	Comments
	Test	Storm	East	West		
28 (F) 29 (E)	X		131,100		572.1	Replacement of Ball Float. Elev. of Tank Top 569.1
28 (F) 29 (E)	X			83,950	572.1	Replacement of Ball Float. Elev. of Tank Top 564.8
Sept.					Monthly Max. 572.8	
6 (F) 7 (E)	X			93,600	571.8	
7 (F) 8 (E)	X		164,900		571.8	
8 (F) 9 (E)	X			174,200	571.9	
16 & 17		X	Not Available	Not Available	Max. 572.8	35,000 Gal. Into Tank. Tanks emptied when Water Level Reached 572.8

TABLE 8
TANK STORAGE DATA

Date	Filling		Volume Stored (Gal)		- Lake Level -	Comments
	Test	Storm	East	West		
22 (F) 23 (E)	X			106,400	571.5	
23 (F) 24 (E)			155,600		571.7	
Oct.					Monthly Max. 571.4	
9 &10 (F) 10 (E)	X		126,000		570.7	
8&9 (F) 10 (E)	X			149,000	570.7	
29 &30 (F) 31 (E)	X		116,000		571.4	
29 &30 (F) 31 (E)				101.000	571.4	
TOTAL			2,684,000	2,141,000		

NOTES:
(1) Design elevation for tank operation 571.9
(2) Overflow invert 572.3 in connection chamber.

TABLE 9

SUMMARY OF TANK STORAGE DATA

	East Tank (Rubber Bottom)	West Tank (Steel Bottom)
Storm Events	5	6
Gallons	496,000	492,000
Tests	14	16
Gallons	2,188,000	1,649,000
Total Gallons Stored	2,684,000	2,141,000

Underwater Storage Tanks

The two underwater storage tanks installed in the pilot facility in Sandusky, Ohio functioned as designed. A few minor problems were encountered with the supporting tank equipment but no operational difficulties were actually encountered with the basic tank itself.

The west tank bottom was manufactured with 1/4" steel sheet. This appears to be the better bottom design due to economics and ease in determination of volume, tank shape, and tank top action. However, the east tank with its rubber bottom functioned properly and might be more applicable for wet installation of the underwater storage system. Dry installation would favor the steel bottom tank design.

The flushing system was included to aid in controlling sedimentation in the underwater tanks. Use of this system in future storage tanks would depend on the physical and chemical properties of the liquid to be stored. In general, the flushing system might be deleted from future underwater storage tanks.

During the operation period, the need for check valves in the influent lines to the tanks was indicated. During a storm event on September 16 and 17, 1969, the tanks were filled with about 35,000 gallons of storm overflow. With a high lake level of 572.82, the storm overflow was forced out of the tank through the influent pipe and back through the leaping weir to the interceptor.

This occurred due to the differential head available, the low leaping weir regulator elevations which exist, the extreme water level fluctuations, and the influent control valves not being designed to close with a partially filled tank.

During a test filling of the east tank on January 1, 1969, a total of 190,000 gallons of water were pumped from the tank. Since the operator had estimated that about 120,000 gallons had been pumped into the tank, it seemed that a problem had developed. The east tank could hold such a quantity at about seven percent fabric elongation. The high pressure pump capacity was erratic in pumping due to its application. However, it was decided that the pressure relief valve should be checked to see if it was open. The west tank functioned properly during this testing.

On February 15 and 16, 1969, the east tank was again filled with bay water and 140,000 gallons were pumped out. On February 16, 1969, the operator noted that water was coming out of the influent pipe to the east tank in the connection chamber. On February 17, 1969, the operator pumped 30,000 gallons from the east tank. The east tank was not empty and the lake level was forcing bay water out of the tank.

Early explanations were that either a leak had occurred in the east tank and its associated piping or the pressure relief valve was open. On February 27th, two divers found the relief valve open. The springs were caught on the rods used to guide the top plate. The pressure relief valve was closed and the sewage pump was started to empty the tank. After the tank was empty (tank height being monitored by the T.L.C.S.), the pump still produced water with large amounts of silt and clay. (This problem was not explained until June, 1969. The gas vent valve float ball had partially failed during the period and did not seal under partially filled tank conditions.) Divers then checked the tanks but found no leaks. A fine layer of silt covered the tank and the visibility was very poor. Further investigations were necessary.

A decision was made to fill the east tank to capacity and then add dye. This was done on April 9, 1969 after the ice had left the bay, and 90,000 gallons of bay water were pumped into the tank and one pound of die was added. On April 10, 1969, divers inspected both underwater storage tanks. Due to the silt covering the tank and the wave action, visibility was negligible. No leaks were found. No dye was found in the water.

On April 15, 1969, 120,000 gallons of bay water was pumped into the east tank and then five pounds of yellow dye was added with another 50,000 gallons of bay water. On April 18, 1969 a severe storm event occurred. The west tank was filled with the 170,000 gallons of water and five pounds of dye. No leakage of the dye was found. The east tank was pumped out on April 23, 1969 after divers again checked the tank.

The pressure relief valve might have opened and closed itself for the leakage on February 27, 1969. On April 23 and 24, 1969, the pressure relief valves were removed. Teflon tubing was inserted over the rods to provide a smooth surface. The holes drilled in the bottom flange were oversized and the tubing reduced the excess clearance from these holes.

One of the causes for the opening of the pressure relief valve was the layer of fine silt which had accumulated over the two tank tops. This layer consisted mainly of a grey silt and clay with some pebbles and rocks from the April storm and was from two to six inches deep. A new breakwall near the pilot facility was washed out by this storm adding to the material deposited.

The accumulation of silt created operational problems with the pressure relief valves in filling the tanks and with the T.L.C.S. The silt on the tanks caused additional head of 0.125 to 0.167 feet to be necessary to fill the tanks. With the low head conditions existing at the pilot facility, this created a problem in operation due to the high bay levels and the maximum static head allowed in the connection chamber prior to backing up flow in the McEwen Street sewer. It was decided that the silt would have to be removed. Attempts to get commercial pool cleaning firms to do the work were unsuccessful since their work schedules were filled.

It was also suggested at this time that an explanation of the infiltration of water to the connection chamber might be caused due to malfuctioning of the gas vent valves. It was decided to examine the gas vent valve at the time of silt removal.

On June 24 and 25, the silt was vacuumed off of the two underwater storage tanks. The two tanks were filled to about 200% of their rated 100,000 gallon capacity. The removal was accomplished using a centrifugal pump and an industrial vacuum head capable of pumping one inch solids. Personnel were able to stand on the filled tanks. The cost of silt removal was about $400.00 per tank.

A diver checked the four corner gas vent valves on each tank. No problems were found with any of these valves. However, on inspection of the center gas vent valves, both copper float balls were found to have failed preventing sealing against the top seal seats except under high interval tank pressures. Also corrosion of the aluminum guides for the float balls allowed the balls to hang up for short periods.

The center copper float balls were replaced, then on August 8, 1969, these were replaced with float balls made of 20 gage stainless steel. No further infiltration was noted through the end of the operation period.

The corner gas vent valves do not experience the severe piston type action of the center gas vent valve and stronger float balls were not necessary. The corner gas vent valves need not be installed in future underwater storage tanks with this shape. A single center gas vent valve can do the required job.

Tank Level Control System (T.L.C.S.)

In order to provide a system to monitor tank volume, the T.L.C.S. has to function consistently in the bay environment. Several problems were encountered in the operation of the system due to errors in operating procedure and the bay environment with its associated waves and silt.

Prior to the installation of the underwater storage system, the bubbler type depth measurement system had not been used except in standard industrial and municipal projects. In these applications, the liquids monitored were usually flowing in one direction while fluctuations in liquid level were steady without waves. While solid particles were encountered in these installations, a deposit of these particles over the bubbler was easily prevented by bubbler location or periodically flushing them away.

The air supply for the T.L.C.S. was sized for the system using the quantity it would require in a typical application for a basis of design. The volume of air was increased by 50% over this. However, after initial operation of the system, it was soon established that the air requirements were much in excess of those provided. A second compressor of 3/4 h.p. capacity was installed in series with the first 1/3 h.p. unit to provide an adequate air supply.

The situation of air supply was compounded when air was required to be released through the bubbler lines on the top of each tank and at the reference elevation at a faster rate than was intended during the operation period. This increase in air feed was required due to the accumulation of fine silt over the bubbler lines. Better T.L.C.S. system accuracy was accomplished with the use of the higher air feed rate.

After the first month of operation, fluctuations were noted which were not entirely representative of actual events as the internal bellows were subjected to a pressure higher than operating pressure. The bellows were strained to a point outside of their range of calibration. With the zero to 23 foot range of operation this distortion and the time required for the bellows to return to calibration required two changes.

The first change was the elimination of higher pressures. During normal operation of the T.L.C.S., the operator at the pilot facility forced any water out of the bubbler lines and cleared silt away from the bubbler lines by forcing 20 psi air through these lines. However, the bellows were also subjected to this 20 psi pressure while operating at 3 to 15 psi. It was found that by eliminating this 20 psi pressure, the drift or fluctuation of the recorded data due to straining of the bellows was eliminated. The T.L.C.S. as installed allowed isolation of all transmitters while blowing out the bubbler lines with the 20 psi air.

The second change made was to use 100% of the 0 to 23 feet bellows range. Operation had been tried using the 0 to 23 feet range at 50% to 30% proportional band and 0 to 80 inch range at 100% proportional band to provide more accuracy.

However, the tracking of the bellows was found to be nonlinear during calibration procedure. No bellows are available between the 0 to 23 foot range and the 0 to 80 inch range. The bellows for the pilot facility should have an optimum range of 0 to 120 inches. The 0 to 23 foot range does not allow as accurate a control as a 0 to 120 inch range would.

This second change allowed the best control of the liquid to be stored in the tank. Under the most severe storm events, the liquid caught by the tanks was 50,000 to 80,000 gallons higher than that during other storm events. This was due to the fluctuation of the tank top during storm events. Since the tank top height was used to determine the tank volume and the point at which the influent control valve closes, tank top elevation fluctuation created by wave action along with the slight insensitivity and time delay in the pneumatic system allows a larger fluctuation in tank capacity.

The fluctuation in tank capacity was not critical in the original tank design of 100,000 gallons since design was based on zero fabric elongation. However, designs incorporating a 6% fabric elongation for future installations will require the more positive control of the volume of liquid storage provided.

Another problem encountered in the operation period consisted of the deposits of silt which accumulated over the bubbler tubes. These silt deposits caused an inaccurate reading due to the increased weight of material causing higher erroneour pressure readings. The major portion of this problem was experienced at the reference bubbler attached at the tank frame. Silt accumulation of 1/2 to 1 inch did not cause difficulties; however, larger accumulations did. Increases in the rate of feed helped reduce this difficulty and periodic removal of silt around the bubbler was required.

The last problem encountered with the T.L.C.S. during the operation period was a leaking air valve prior to the three-way switching valve which controls the 12-inch Vee-Ball influent control valve. The 12-inch Vee-Ball valve is a positive control valve in that when air supply is lost the valve closes. The three-way switching valve operates such, that when the tank is filled, on a signal from the transmitters the air supply to the valve is shut off and the line to the valve is opened to atmosphere, closing the valve. Since the valve is open at 20 psi and closed at about 3 psi, if a pressure greater than 2 psi is applied to the line by a leaking air valve, the valve opens slightly.

Two times in July, 1969 storage data was lost when air leaks occurred after the tanks were filled and the valves opened partially. With bay levels being higher than other elevations in the storage system and the McEwen Street regulator, the stored overflow was forced out of the tanks through the influent piping and back through the leaping weir to the interceptor. The stored liquid did not got into the bay. This system was corrected by attaching an additional regulator after the three-way switching valve. After the valve is closed and air pressure drops past 15 psi, the regulator opens the line to the control valve to atmosphere. Before the control valve can be opened again, the additional regulator must be closed manually.

Flow Measurement

The system used to monitor dry-weather flow from the drainage area and to record storm overflow to the connection chamber incorporated Leupold & Stevens Model S61R ball float type transmitters with remote recorders. The dry-weather transmitter was calibrated to measure flow through an existing 12-inch wide concrete channel in the existing regulator-over-flow device. The storm overflow transmitter was used in conjunction with a special 30 inch Leupold-Lagco flume.

The transmitters were both mounted in existing regulator chambers. These components were subjected to the atmospheric conditions encountered in combined sewers. The problems encountered during the period of operation consisted mainly of incomplete sealing of the transmitter housing. When this occurred, internal mechanical and electrical parts corroded. Corrections of this problem required replacement and cleaning of parts and complete sealing of both units.

The Leupold and Stevens recorders for the transmitters also required attention. The signal from the transmitter to the recorder was sensitive to electrical interference. When first installed in the control building, the flowmeter wiring was not shielded properly. This caused excessive chattering and wearing of internal gears which needed to be replaced. Shielded wire was installed and isolation transformers on the incoming power helped reduce this problem.

The friction feed of the chart paper in the recorders caused many difficulties in obtaining data on a correct time basis.

Table 10 shows periods of time in which the flow measurement system was not operating during the operational period with reasons for the inoperation.

The Leupold and Stevens units were in operation from March, 1968. They were originally housed in a temporary metal building. The units were transferred to the control building in October, 1968.

The Fischer-Porter magnetic flowmeter monitoring storm overflow pumped from the storage tanks operated consistently throughout the operational period. No difficulties with this unit nor its recorder were encountered.

Flooding of Interceptor

An unexpected difficulty encountered during the one-year of operation, which was not anticipated from site investigations prior to construction, was, surcharging of the interceptor

TABLE 10

FLOWMETER OPERATION

Date	Dry-Weather Flow		Storm Overflow Flow		Comments
	Transmitter	Recorder	Transmitter	Recorder	
1968 Nov.		Excessive chattering		Excessive chattering	Company Contacted
Dec.		Excessive chattering and wearing of gears		Excessive chattering and wearing of gears	Shielded wire installed.
1969 Jan.		Recorder chart mechanism broken Jan. 2 - 24 (pen not operating continously)		Chattering	Potentiometer replaced and transmitter sealed.
Feb.					Isolation Transformers installed Feb. 14.
March					

TABLE 10

FLOWMETER OPERATION

Date	Dry-Weather Flow		Storm Overflow Flow		Comments
	Transmitter	Recorder	Transmitter	Recorder	
1969 April	Potentiometer corroded - Erratic Operation			Parts broken on April 19 Out of operation	
May				Repaired on May 27	
June	Repaired Potentiometer June 20	Replaced gears and potentiometer June 9			
July					
Aug.		Ink pen broken Aug. 31.			
Sept.	Potentiometer corroded out Sept. 18 replaced Sept. 24.	Ink pen replaced Sept. 18			

TABLE 10

FLOWMETER OPERATION

Date	Dry-Weather Flow		Storm Overflow Flow		Comments
	Transmitter	Recorder	Transmitter	Recorder	
1969					
Oct.	Oct. 21 to 27 -- Electricity to both units shut off at panel Potentiometer corroded and gears corroded Removed Oct. 27				
Nov.			Removed Nov. 5 due to corroded shafts		

sewer. Part of this problem was due to the extremely high water levels encountered from April, 1969 to September, 1969 and the poor tide gates and low regulator - overflow elevations existing in the combined sewer collection system.

At the McEwen Street regulator-overflow device, the interceptor to the water pollution control station was flooded at times by bay water flowing back through the Ogontz Street outfall and through the Ogontz Street regulator into the interceptor. When the City of Sandusky completed installation of nine new tide gates, including one at the Ogontz Street outfall in July, 1969, flooding of the interceptor with bay water was reduced.

Upstream from the McEwen Street connection to the interceptor, two contributing drainage areas feed the interceptor. The first drainage area immediately east is the Arthur Street drainage area which consists of about 323 acres drained by combined sewers. The regulator device at the Arthur Street connection to the interceptor is a No. 3 Brown and Brown automatic sewage regulator which has a rated capacity of 3.75 CFS at 0.1 feet of head and 11.90 CFS at 1.0 feet of head with an open gate. Actual hydraulic capacity and operation of this regulator was not determined. It is expected that the regulator closes with a head of three to four feet.

The second drainage area is controlled by the Farwell Street pump station which pumps sanitary sewage from the sanitary sewer collection system east of the Arthur Street drainage area and Cedar Point. Pump capacity at the Farwell Street pump station is 13.5 MGD (20.88 CFS). This maximum flow rate occurs periodically during five minute intervals when pumping the wet well down. Three pumps are used to supply this flow rate. Lesser flow rates occur over longer intervals supplied by one or two pumps.

The interceptor capacity above the McEwen Street connection is about 25.0 cfs maximum and about 23.5 cfs when flowing full. Beyond the McEwen Street connection, the interceptor capacity is about 28.0 cfs maximum and 27.0 cfs when flowing full.

During storm events, the Arthur Street drainage area contributes 0 cfs to a peak of over 11.9 cfs and the Farwell Street pump station contributes peaks of 20.88 cfs. This total flow is in excess of the interceptor capacity. When the interceptor was flooded with bay water by high water levels during storm periods, surcharging of the interceptor by the Farwell Street pump station caused water to be forced up through the McEwen Street regulator device. This created problems in gathering representative samples of combined sewer storm overflow and in measurement of both dry-weather flows and storm overflows.

Investigation of blueprints of existing sewage collection systems and discussion of the operation of the system in the area of the pilot facility with the sewer maintenance crews did not indicate the possible surcharging at the interceptor by the Farwell Street pump station.

Electrical Power Costs

One of the major operational costs during the one-year operation period for the pilot facility was electrical power costs. The relative high cost for power is due to the high load demand put on the power supply for short periods of time.

The pilot facility required power supply for the following items:

1. Thirty HP electric motor on sewage pump;
2. Fifteen HP electric motor on high pressure pump;
3. Three, 1 HP electric motors on miscellaneous pumps;
4. Electric lighting, water heater, and baseboard heaters;
5. Flow meters and recorders; and
6. Miscellaneous solenoid valves and laboratory equipment.

The power requirements for the facility were 120/208 volt three phase service for which lines had to be extended from existing Ohio Edison Company lines. Since the pilot facility was considered a commercial establishment, it had to be on a demand meter. The net monthly billing load in KW was based on 60% of the highest billing load during the preceding eleven months or the measured load for the month whichever was greater, as determined by highest 30 minutes load recorded by the demand meter.

After the first three months of operation, the procedure used in operating the pilot facility during emptying of the underwater storage tanks was to supply all electric power with the auxiliary power system except for the 30 and 15 HP pump motors during tank emptying to reduce this peak demand. Both pumps were not operated at the same time. But were alternated as desired. This peak demand was experienced only on an average of four hours per month. Since the billing was based on 60% of the highest billing load for the preceding eleven months, no larger reduction in power cost was experienced during the one-year of operation.

Future installations should be designed to exert a minimum power demand on the power supply. Trickling pumps might be used where emptying of storage tanks over a short time is not necessary. Determination of the maximum and minimum time available to pump stored liquid from the storage tanks must be accomplished before operation costs for underwater storage systems can be determined.

Operation Manual

An operation manual for the pilot facility in Sandusky, Ohio was required during the beginning of the operation period. The operation manual was submitted with corrections on February 14, 1969 and accepted by the F.W.P.C.A.

Operational procedures were explained in detail. Major items included were:

1. Analysis of storm water runoff;
2. Storage Operation;
    a. Filling during storm event
    b. Filling without storm event
    c. Emptying storage tanks,
    d. Piping valve location and operation, and
    e. Pump operation and maintenance;
3. T.L.C.S.;
4. Flow measurement,
5. Auxiliary power and electricl system; and,
6. Rain gage operation.

Manufacturer's operation and maintenance manuals were used in conjunction with this operation manual.

Safety

During November, 1968, a security fence was installed around the pilot facility control building and connection chamber. The fence was added to prevent injury to children who play in the area and to reduce possibility of vandalism.

On January 2, 1969, a letter was sent to the offices of Rohrer by the City of Sandusky requesting the installation of a fence along the bay shore westward from the existing fence and parallel to the railroad tracks and also northward from the existing fence along the bay shore. The fence was to extend to the limits of the deeper water covering the underwater storage tanks. Parents in the area of the pilot facility had voiced an opinion of the possibility of children playing in the deeper water over the tank area. Prior to the pilot facility installation, water depth was only two to four feet deep in this area.

A difficulty in installation of this fence existed due to the rights of free access to the bay of landowners adjacent to the bay. If this additional fencing had been installed, free access would have been impaired. Legal action against Rohrer and the F W.P.C.A. could be brought. Additional fencing might have prevented help from reaching any child in distress.

The area surrounding the pilot facility is used each winter to park ice-boats. Additional fencing would have prevented present use and might have contributed to public rejection of the system.

A letter of explanation was sent to the City Manager on January 6, 1969 requesting that the City should bear responsibility for any litigation by adjacent landowners arising from installation of the fence. The matter was dropped.

It does not seem feasible at this time or any future time that all shoreline can be fenced at future installations. Installation of system pumps and controls in prefabricated underground pump stations would tend to reduce the need for any fencing and would probably be more acceptable by adjacent landowners.

Underwater storage tanks for this system can in many instances be placed in deeper water or further from the shore. However, system costs and installation costs may be lower with near-shore tank location where possible.

Three signs warning of deep water had originally been installed around the tank area during December, 1968.

SECTION VIII

EVALUATION

To evaluate the feasibility of the underwater storage system, the reduction in pollution load from the combined sewer must be determined and the cost of the system must be determined. To evaluate the best approach for pollution abatement, a comparison of the reduction of pollution provided and a comparison of costs on a similar basis must be accomplished for each combined sewer drainage area.

To determine the most economical approach to combined sewer pollution abatement, the entire range of methods, sewer separation, storage, treatment, and storage with treatment, must be examined. The degree of treatment provided by each along with the degree of pollution abatement necessary must be known. In most cases, use of several methods of pollution control will produce the most economical pollution abatement method for a combined sewer system.

Elimination of 100% of all combined sewer storm overflow is usually impractical unless unlimited funds are available. This is due to the large volume of runoff produced by natural events. During the year of operation of the pilot facility in Sandusky, Ohio a storm event on July 4th and 5th yielded 5.57 inches of rainfall over an eight hour period. The average intensity was 0.70 in/hr with intensities experienced over 3 in/hr. Estimated runoff for the McEwen Street drainage area of 14.86 acres was a minimum of 1.46 million gallons. Estimated runoff from the Sandusky combined sewer area of 2205 acres was a minimum of 216 million gallons. This storm is estimated as a 75 to 100 year storm event.

During the year of operation, storm events of 1.84 inches on December 28 and 29, 1968 and 1.61 inches on July 17, 1969 have according to present recurrence curves a return period in excess of the preliminary design storm. Runoff from the two events was estimated at 330,000 gallons and 250,000 gallons respectively. Tank capacity during the first storm was set at 100,000 gallons each. Tank capacity during the second storm was in the process of being increased from 100,000 gallons to approximately 150,000 gallons.

Storm Interception

The design storm used to size the capacity of the pilot facility was a one-year storm of sixty minute duration. The

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related intensity seemed to be 1.30 in/hr from preliminary analysis of excessive short duraiton rainfall. Using the 1.30 in/hr intensity and a weighted runoff coefficient of 0.42 to determine expected storage capacity of the underwater storage tanks, a capacity of 218,000 gallons would be needed. A basic tank capacity of 100,000 gallons was used for design of each tank. This was provided at zero fabric elongation.

In 1969, intensity - duration - frequency curves were constructed from excessive short duration rainfall for 1942 to 1964 from Sandusky WB station data. These curves indicate an intensity of 1.0 in/hr corresponding to the one-year storm of sixty minute duration. Assuming a weighted coefficient of 0.42, the resulting storm runoff volume would be about 168,000 gallons.

Using the 200,000 gallon basic tank capacity, and the intensity - duration - frequency curves, the pilot facility would intercept a two-year storm of 60 minutes duration with a corresponding intensity of 1.20 in/hr. The runoff volume was determined using the 0.42 weighted runoff coefficient.

Since zero fabric elongation was used as a basis of the 100,000 gallon tank capacity design, additional storage capacity exists in the pilot facility tanks. When the underwater storage tanks were designed, no data was available on the use of neoprene rubber coated nylon fabric for underwater storage of combined sewer overflows; and, the manufacturer would not recommend working design fabric elongation for the proposed environment.

After the completion of the operation period and the observation of the fabric in operation in the underwater environment, it is recommended that a six percent elongation be used as a maximum elongation for the tank design and fabric to be used. The use of six percent fabric elongation creates additional storage capacity of about 80,000 gallons in the west tank and slightly more in the east tank. Therefore, existing tank capacity, which can be used at the existing facility, is about 360,000 gallons.

Using the intensity - duration - frequency curves, the 360,000 gallon capacity will intercept a ten-year storm of 100 minute duration with an associated 1.25 in/hr intensity. This runoff volume was calculated using the 0.42 weighted runoff coefficient. All storms with a more frequent return interval will be intercepted unless flow rates to each storage tank is exceeded. This will occur at intensities in excess of 2.00 inches per hour with durations equal to or greater than the time of concentration for the drainage area.

One part of the work which was to be accomplished during the year of operation was the determination of the runoff coefficient for the drainage area versus preceding rainfall, intensity of rainfall, and duration of rainfall. Since the flowmeter installation did not operate properly sufficient data was not obtained for this determination. However, for moderate intensity and duration storms such as a one-year storm, a runoff coefficient of 0.32 to 0.35 would more closely fit the drainage than the 0.42 design coefficient.

Costs

The construction costs for the pilot facility are given in Table 11. Major costs which might be avoided in the future are $57,000 which was required for rock excavation and $30,489 which was the additional cost of the east tank over the west tank. It should be possible to avoid extensive rock excavation at future sites in Sandusky, Ohio by reducing the depth of water over the storage tanks at a filled condition. The additional material cost and fabric installation costs of the bottom fabric of the east tank reduces the east tank's desirability from an economic aspect. Other cost reductions can be achieved in future non-research installations by reducing instrumentation and recording equipment and using an underground prebuilt pump station to house all pumps and controls.

TABLE 11

Pilot Facility Construction Costs

Site Preparation (Dry Installation)	$128,217
West Tank (Steel Bottom)	63,631
East Tank (Rubber Bottom)	94,120
Connection Chamber and piping	19,200
Control Building and pumps	38,230
Instrumentation	29,175
Miscellaneous Construction	4,055
TOTAL	$376,628

Table 12 shows a comparison of the cost of the pilot facility at original design capacity to existing storage capacity at allowable six percent fabric elongation. These costs are extremely high per gallon due to the added costs arising from rock excavation, instrumentation, and control building.

TABLE 12

Construction Cost Per Gallon of Storage Capacity
Pilot Facility Sandusky, Ohio

	$/Gallon
Design Capacity 200,000 Gallons	1.88
6% Elongation Capacity 360,000 Gallons	1.04

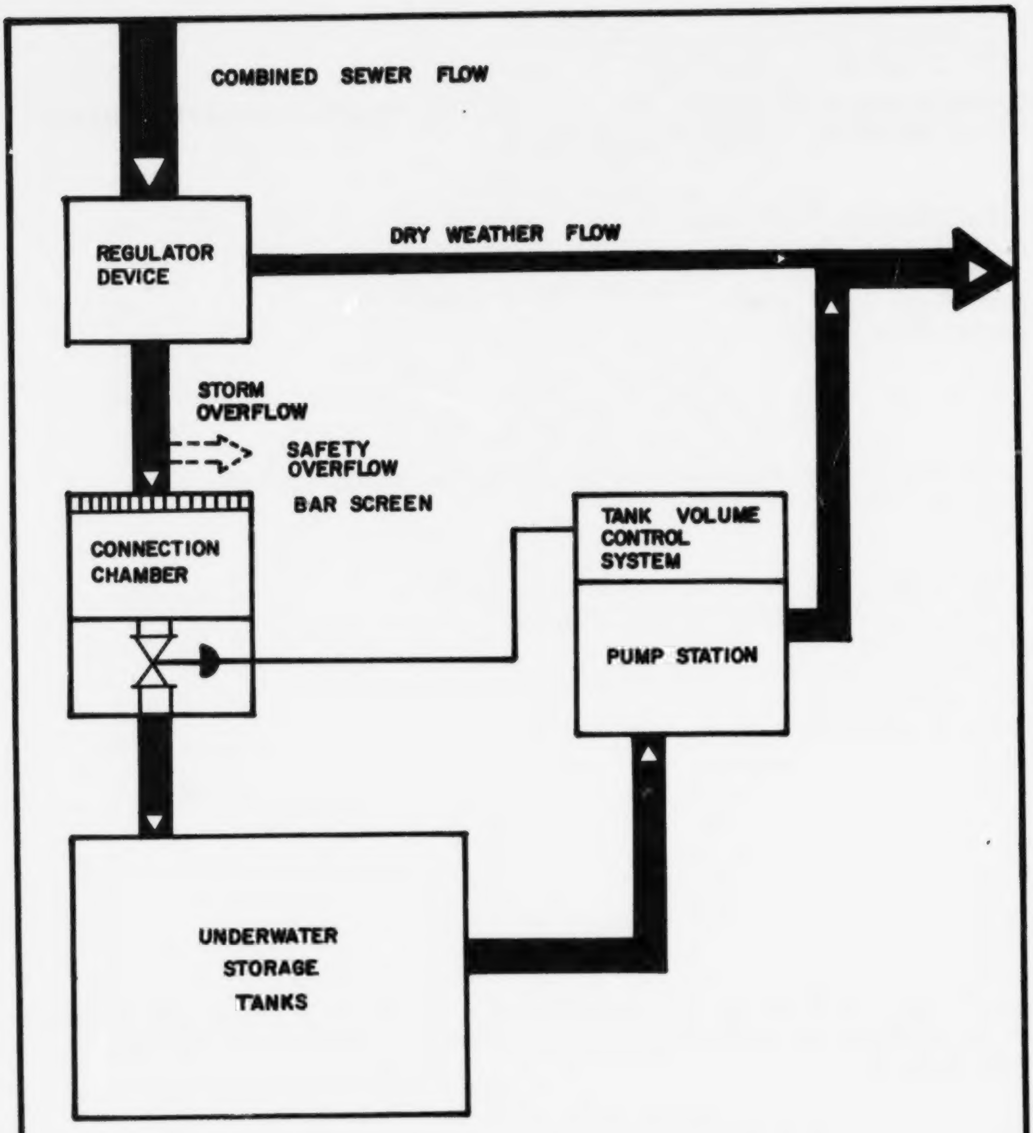
Future underwater storage installations without the data gathering capabilities of the existing pilot facility can be built at a lower cost per gallon of storage capacity. Estimates by Rohrer of underwater storage cost based on present tank design are shown on Table 13. A cost of 42.5 cents per gallon could be achieved at a 200,000 gallon storage capacity and a cost of 35.9 cents per gallon could be achieved at a 1,000,000 gallon storage capacity. Future installations would consist of:

1. Basic storage tank, with pressure relief valve, sedimentation chamber, and gas vent valve;
2. Modified connection chamber with bar screen and safety overflow;
3. Underground prefabricated pump house for pumps and controls; and
4. Tank volume control system.

Additional cost reductions could be achieved at favorable locations with sufficient available hydraulic head, sufficient water depth and minimum wave forces, and modified tank design for wet installations. The tank volume control system would operate from fabric elongation. Figure 39 shows a flow diagram for simplified future installations.

Use of 8% to 12% working fabric elongation with maximum elongation of 18% is possible after more operational data is secured. This would increase existing tank capacity from the 180,000 gallons at 6% elongation to 210,000 gallons at 8% elongation or 223,000 gallons at 12% elongation.

Table 14 shows projected construction costs for a 200,000 gallon storage capacity incorporating these design changes.


COMBINED SEWER FLOW
REGULATOR DEVICE
DRY WEATHER FLOW
STORM OVERFLOW
SAFETY OVERFLOW
BAR SCREEN
CONNECTION CHAMBER
TANK VOLUME CONTROL SYSTEM
PUMP STATION
UNDERWATER STORAGE TANKS

TYPICAL INSTALLATION
TEMPORARY UNDERWATER STORAGE OF COMBINED SEWER
STORM OVERFLOW

KARL R. ROHRER ASSOCIATES, INC.
AKRON, OHIO

FIGURE 39

TABLE 13

Construction Cost for 200,000 Gallon Storage Capacity Based on 6% Working Fabric Elongation

Site Preparation (Wet Installation)	$13,700
Tank (Steel Bottom)	54,630
Connection Chamber and Piping	8,600
Pump Station	6,500
Instrumentation	1,500
	$84,930

Cost for 1,000,000 Gallon Storage Capacity Based on 6% Working Fabric Elongation

Site Preparation (Wet Installation)	$ 43,500
Tanks (Steel Bottom base 200,000 gallon capacity)	273,150
Connection Chamber and Piping	27,020
Pump Station	7,500
Instrumentation	7,500
	$358,670

TABLE 14

Construction Cost for 200,000 Gallon Storage Capacity Based on 8% Fabric Elongation, Modular Tank Design, and Minimum Wave Force

Site Preparation (Wet Installation)	$13,700
Tank	29,570
Connection Chamber and Piping	8,630
Pump Station	5,800
Instrumentation	1,200
	$58,900

The cost would be about 29.4 cents per gallon of storage capacity.

Land costs are not included in the cost figures given. In cases where property or right-of-way has been obtained for sewer lines or property is already owned for other reasons, additional system costs for land to install the pump station on will be negligible. Where any appreciable amount of land has to be purchased, cost per gallon of storage will reflect this cost.

Operation costs for the pilot facility in Sandusky, Ohio were far in excess of future systems. Future systems should be relatively maintenance free and require checking by maintenance personnel only after major storm events and at regular intervals.

Estimated minimum operation costs for the Sandusky, Ohio pilot facility are about $4,500 per year. Variation in this minimum cost will vary greatly with the cost of electricity or other power supply for the pumps used. The pump capacity and the minimum allowable time available to pump out the storage tanks will need to be determined prior to accurate estimates of operation costs for future installations.

Monthly testing of the underwater tanks is unnecessary and uneconomical. However, testing of the tanks by filling with bay water should be done annually.

Table 15 shows a comparison of the cost of storage capacity for the underwater storage tank compared to underground concrete tanks or lagoons. The projected cost does not include preliminary studies for each site, extra costs encountered due to specific site conditions, land acquisition, acquisition of all permits, and major alterations to existing structures. Operation costs are not included.

With the cost comparison figures in Table 15, it can be seen that underwater storage can readily compete with other types of storage where applicable. Land costs which increase the cost per gallon of storage will not be as great as for other types of storage. Many times land will not be available. Aesthetics for the underwater and underground tank are comparable. However, open lagoon storage is a poor method to be used in populated areas. Public health regulations may prohibit this type of storage.

TABLE 15

Storage Capacity Construction Cost Comparison

(Based on Costs for Sandusky, Ohio Area)

Storage Method	Storage Capacity In Gal.	Construction Cost in $	Cost/Gal in ¢/gal
1. Underwater tank	200,000	84,930	42.5
	1,000,000	358,670	35.9
2. Reinforced Concrete Tank	200,000	95,820	47.9*
	1,000,000	351,740	35.2*
3. Lagoons	200,000	22,800	11.4*
	1,000,000	102,000	10.2*

Note: Engineering studies and design, land cost, acquisition of permits are not included in cost figures.

* Solids removal is not included in these cost figures.

For the Sandusky, Ohio combined sewer system, Rohrer completed preliminary construction costs for several methods of combined sewer pollution abatement. These estimated costs are shown in Table 16 along with cost estimates from other sources for similar or other pollution abatement methods.

Comparison of the underwater storage with partial storage or in-system storage systems or partial treatment systems is meaningless unless the degree of pollution reduction is known.

Storage alone is insufficient without a minimum of primary treatment and chlorination. Secondary treatment of storm flow would present difficulties where biological secondary treatment is used, and phosphate removal might be necessary. No costs can be determined for added treatment capacity until minimum pumping periods of the stored overflows are determined.

Expansion of existing treatment facilities would depend on what time would be allowed for emptying all storage tanks and the DWF to the treatment facility compared to design flow. If stored overflow could be pumped to the treatment facility over longer periods (three days to a week), treatment facilities could possibly handle the flows at non-peak hours (during the night and over weekends) In this manner, increased capacity might not be required.

TABLE 16

COMBINED SEWER POLLUTION ABATEMENT COST COMPARISON

Location and Method	Area Served In Acres	Cost ($ x 10^6)	Cost Per Gal (¢/Gal.)	Cost Per Acre (4/Acre)	Cost Per Capita ($/Capita)
I. Storage					
1. Sandusky, Ohio Underwater Storage (1 year storm)	580	3.57	40.0	6160	----
2. Sandusky, Ohio Underwater, underground, and surface (1 year storm)	2205	7.75	26.1	3500	212
3. Chicago, Illinois[2] Chicago Deep Tunnel (100 year storm)	192,000	1,270	----	6615	423
4. Saginaw, Mich.[1] Lagoons	----	----	----	415 to 5140	62 to 670
5. National[1]	----	----	----	20 to 6500	6 to 1300

TABLE 16

COMBINED SEWER POLLUTION ABATEMENT COST COMPARISON

Location and Method	Area Served In Acres	Cost ($ x 10^6)	Cost Per Gal (¢/Gal.)	Cost Per Acre ($/Gal.)	Cost Per Capita ($/Capita)
II. Treatment (Primary with Chlorination)					
1. Sandusky, Ohio Increase in plant and interceptor capacity (1 year storm)	2205	17.2	----	7800	472
III. Sewer Separation					
1. Sandusky, Ohio No. plumbing changes in individual structures	2205	30.6	----	13,878	839
2. Sandusky, Ohio Complete	2205	44.1	----	19,998	1208
3. Midwest[1] Complete	----	----	----	100 to 29,000	25 to 2,660

TABLE 16

COMBINED SEWER POLLUTION ABATEMENT COST COMPARISON

Location and Method	Area Served In Acres	Cost ($ x 10^6)	Cost Per Gal (¢/Gal.)	Cost Per Acre ($/Gal.)	Cost Per Capita ($/Capita)
4. National[1] No Plumbing changes	----	30,400	----	1020	835
5. National[1] Complete	----	48,800	----	16,320	1300

[1] U.S. Department of the Interior, Federal Water Pollution Control Administration, "Problems of Combined Sewer Facilities and Overflows, 1967 "Water Pollution Control Research Series, No. WP-20-11, by the APWA, December 1, 1967.

[2] Bauer, William; Dalton, Frank E., and Koelzer, Victor, The Chicagoland Deep Tunnel Project, Metropolitan Sanitary District of Greater Chicago, September, 1968.

Investigation of the pollution load from combined sewer drainage areas must be conducted to determine allowable times over which stored overflow can be pumped out. Determination of the length of time for significant buildup of deposits of sanitary wastes in combined sewers should be made.

Separation of sewers may not be the economical answer for elimination of combined sewer pollution load. From studies conducted during the past decade, evidence is that the total pollution load from storm overflow from combined sewer systems is not from sanitary flow only. About 80% of the pollution load is due to the sanitary wastes. The other 20% of the pollution load is entirely due to storm runoff. References 2, 3, 4, and 25 in the Bibliography of this report indicate this pollution load. Further studies should be made to further define the extent of the pollution load from storm sewer systems.

Storage Capacity and Pollution Reduction

Temporary storage of storm overflow from combined sewer systems is a method proposed to reduce water pollution. The principal mechanisms of this pollution reduction are containment and delay. Combined sewage which would normally overflow to the receiving waterbody is contained within the storage facility where treatment is delayed until such time that the normal sewage treatment plant has the capacity to process it. It can be seen that the pollution reduction of such a system is a combination of the percentage of the pollution load contained and the degree of treatment afforded at the sewage treatment plant.

Certainly other factors complicate the problem such as interceptor capacity, strength and freshness of the stored overflow, treatment plant capacity, and frequency of recurring storm events. However, under the most optimum conditions, the maximum pollution reduction which would be possible is the product of the pollution load contained and the degree of treatment at the sewage treatment plant. If 100% of all overflow from any storm or combination of storms could be contained, the problem would simply be the treatment afforded by the sewage treatment plant. It can be seen that the storage capacity is directly related to the pollution reduction capability of such a storage system.

As storage capacity is increased to intercept larger storms, however, the cost of pollution containment increases disproportionately. At some point it becomes more expedient for the cost of pollution reduction to apply the funds toward improving the degree of treatment at the sewage treatment plant.

Advocates of partial storage systems point out that the major pollutant load encountered in storm overflow from combined sewers occurs during the first flushing action of a storm. If storage can be provided to accomodate this first flush of water, a high degree of pollution control will be accomplished at much less capital investment than with sewer separation or total storage systems.

It is understood that partial storage systems will not provide 100% capture of storm overflow from combined sewers. However, if the initial overflow from each storm may be captured, the excess overflows will be much less deleterious in water quality.

As an example, consider the storage of one inch of rainfall in the Akron, Ohio area. For 83 years of record, 87 separate storms occurred producing more than one inch of rainfall. If the rainfall in excess of one inch were compared to the total rainfall for this period, it amounts to only 4.2%. If it can be assumed that 20% of the total combined sewer load is due to storm runoff, than only 0.84% of the total runoff pollution would overflow the one inch storage facility. Consider also the duration of overflow events corresponding to storm events in excess of one inch. Again for the 83 years of record, overflows occurred 0.31% of the time. If it can be assumed that 80% of the pollutional load is attributed to sanitary sewage, then 0.25% of the total sanitary sewage pollution would overflow the one-inch storage facility. It is evident therefore, that only about 1.1 percent of the total combined sewer pollution load could escape a one-inch rainfall storage facility.

Even considering the severe storm experienced July 4th and 5th during the year of operation, the Sandusky Pilot Facility retained 59% of the overflow which would have normally been discharged directly to Lake Erie. Overflows from the facility occurred approximately 1% of the time. By applying the 80% factor for sanitary sewage and 20% factor for storm water, the Sandusky Pilot Facility and combined sewer system contained over 90% of the pollution load and delivered it for treatment at the water pollution control station.

Aesthetics

One of the items to be considered in the evaluation of the pilot facility was the aesthetics relationship with the surrounding area. Figure 13 showing the McEwen Street outfall speaks for itself. While doing preliminary investigations, strong odors were present many times in the area of this outfall. After installation of the pilot facility, no odors were noticed.

While the pilot facility control building and fencing fits into the area, the underground prebuilt pump station for future installations should blend with any existing housing scheme. The underwater tanks as well as underground tanks also tend to fit easily into any existing housing scheme.

SECTION IX

ACKNOWLEDGEMENTS

It has required the support and kind assistance of many individuals and organizations to make this project possible. The following acknowledgements are only a few of those to whom we are sincerely grateful.

This project could not have been possible without the funding and support of the Water Quality Office, Environmental Protection Agency. Assistance and direction were provided by the division of Applied Research and Development through Mr. Allen Cywin; the Storm and Combined Sewer Pollution Control Branch through Mr. William A. Rosenkranz, Chief, and Mr. George A. Kirkpatrick, Project Officer; and the Procurement Branch through Mr. Robert L. Wright, Contracting Officer, and Mr. John H. Blake.

The City of Sandusky through the approvals by the City Commission and assistance provided by Mr. Paul A. Flynn, past City Manager and the late Mr. William R. Donahue, City Engineer proved invaluable to the project.

Bay Construction Company of Sandusky, Ohio is acknowledged for the competent construction of the facilities.

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The State of Ohio, Department of Health is acknowledged for the assistance and approvals provided.

Acknowledgement is made for the assistance by various members of the firm throughout the project.

Mr. Syed M. Nehal who made outstanding professional engineering contributions in many aspects of the project.

Mr. Harold L. Laurila whose able assistance in the construction and operation phases was invaluable.

Mr. James F. Forsythe who provided the electrical engineering for the facility.

Karl R. Rohrer

William J. Bandy, Jr.

SECTION X

AWARDS

With F.W.P.C.A. approval, Rohrer entered the demonstration pilot facility in the Ohio Society of Professional Engineer's program, "The Seven Engineering Wonders of Ohio" for 1968. On January 23, 1969, Rohrer was notified that the Sandusky Pilot Facility had been chosen as one of the final choices for this annual award by the Ohio Society of Professional Engineers.

On January 19, 1970, Rohrer was again notified that the Sandusky Pilot Facility had been selected as one of the outstanding engineering achievements for 1969 by the National Society of Professional Engineers. Entrance of State award winners into this program is automatic.

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SECTION XI

PATENTS

On behalf of the U.S. Department of the Interior, Federal Water Pollution Control Administration, and Karl R. Rohrer Associates, Inc., Rohrer has applied for patent rights for the underwater pilot demonstration facility.

The Underwater Tank System patent was issued on March 30, 1971 numbered U.S. Patent No. 3,572,506.

The Automatic Sampling Device patent was issued on June 28, 1971 as U.S. Patent No. 3,587,324.

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SECTION XIII

GLOSSARY

COMBINED SEWER - A sewer which carries sanitary sewage with its component commercial and industrial wastes at all times and which during storm or thaw periods serves as the collector and transporter of storm water from streets or other sources; thus, serving a "combined" purpose. Combined sewers make provision for the overflow of excess amounts of flow from the combined system to points of discharge.

COMPLETE SEWER SEPARATION - Separation of all public combined sewers into two separate and independent sewer systems, one for the handling of sanitary sewage and industrial and commercial wastes and the other for the handling of storm water flow.

DRAINAGE BASIN - A geographical area or region which is so sloped and contoured that surface runoff from streams and other natural watercourses is carried away by a single drainage system by gravity to a common outlet or outlets; also referred to as a Watershed or Drainage Area.

FREQUENCY OF STORM (DESIGN STORM FREQUENCY) - The anticipated period in years, which will elapse, based on average probability of storms in the design region, before a storm of given intensity and/or total volume will recur; thus, a 10 year storm can be expected to occur on the average once every 10 years. Sewers designed to handle flows which occur under such storm conditions would be expected to be surcharged by any storms of greater amount or intensity.

GAS VENT VALVE - Valve to release entrained or evolved gases from interior of underwater storage tanks.

IN-SYSTEM STORAGE - Facilities or the capacity for holding or retaining of flows of sewage and other wastes in subterranean storage chambers or other portions of the sewer system in order to minimize overflows from combined sewers and permit the treatment of large volumes of such flows.

INTERCEPTION RATIO - Pertaining to combined sewer regulators, it is the ratio of the maximum flow which can be directed to the interceptor sewer to the normal dry weather flow.

INTERCEPTOR SEWER - A sewer which receives dry-weather flows from a combined collection sewer system and pre-determined additional amounts of storm flow by means of any form of regulating device and then conducts these flows to point of treatment or discharge. The flow not intended to be so conducted then overflows to receiving waters.

LIMITED BODY CONTACT RECREATION - Use of natural waters, such as rivers, lakes, and coastal waters, for recreational purposes which do not represent deliberate or planned total body immersion such as swimming or bathing; thus, use of waters for boating, fishing, and related sports.

OFF-SYSTEM STORAGE - Facilities for holding or retaining excess flows from combined sewers, over and above the carrying capacity of the interceptor sewers, in chambers, tanks, lagoons, ponds, or other basins which are not a part of the subsurface sewer system.

OVERFLOW - The excess flow from combined sewers which is not conveyed to the plant for treatment but is transmitted by pipe or other channel directly to the receiving waters.

PRESSURE RELIEF VALVE - Protective relief valve preset to prevent overfilling of underwater storage tanks in case of control valve failure.

PRIMARY TREATMENT - Processes or methods, that serve as the first stage treatment of sewage and other wastes intended for the removal of suspended and settleable solids by gravity sedimentation; provides no changes in dissolved and colloidal matter in the sewage or wastes flows.

REGULATOR - A structure installed in a canal, conduit, or channel to control the flow of water or wastewater at intake, or to control the water level in a canal, channel, or treatment unit. A device for regulating the diversion of flow in combined sewers. A device for regulating water pressure.

SANITARY SEWAGE - Wastewater discharged from homes, commercial establishments, and other structures; designated as "sanitary" flow because it is composed of used or spent water resulting from human use in so-called sanitary conveniences.

SANITARY SEWER - A sewer that carries liquid and water-carried wastes from residences, commercial buildings, industrial plants, and institutions, together with minor quantities of ground, storm, and surface waters that are not admitted intentionally.

SECONDARY TREATMENT - Processes or methods for the supplemental treatment of sewage and other wastes, usually following primary treatment, to effect additional improvement in the quality of the treated wastes by biological means of various types, including activated sludge treatment or tickling filter treatment; designed to remove or modify organic matter.

SEDIMENTATION CHAMBER - Chamber used to control solids within underwater storage tank and allow pumping liquid and solids.

SEWAGE PUMPING STATION - An installation that pumps or lifts sewage and other wastes from a lower level in a sewer system into a higher sewer or a receiving chamber for transportation to a treatment plant or point of discharge.

STATIC REGULATOR - A regulator which has no moving parts or has movable parts which are insensitive to hydraulic conditions at the point of installation and which are not capable of adjusting themselves to meet varying flow or level conditions in the regulator - overflow structure.

STORM WATER - The excess water running off from a surface of a drainage area during and immediately after a period of rain. It is that portion of the rainfall and resulting surface flow that is in excess of that which can be absorbed through the infiltration capacity of the surface of the basin.

STORM SEWER - A sewer that carries storm water and surface water, street wash and other wash waters, or drainage, but excludes domestic wastewater and industrial wastes. Also called storm drain.

TIDE GATE (BACKWATER GATE) - A gate installed at the end of a drain or outlet pipe to prevent the backward flow of water or wastewater. Generally used on sewer outlets into streams to prevent backward flow during times of flood or high tide.

WATER POLLUTION CONTROL PLANT - An arrangement of devices and structures for the control of waterborne pollution of waterways. Also referred to as treatment plant, with appropriate adjective describing source of wastewater.

WET-WEATHER TO DRY-WEATHER RATIO - An "indicator" of the capacity of an interceptor sewer, as designed to carry the higher flows resulting from periods of storm, compared to the capacity provided by design to handle flows during dry weather. In an interceptor sewer, the additional wet-weather flow capacity makes it possible for the sewer to carry a predetermined amount of storm water admixed with sanitary flows to the point of ultimate treatment or disposal. Excess storm water is permitted to flow directly into receiving waters at overflow points in the collection sewer system.

SECTION XIV

ABBREVIATIONS

F.W.P.C.A.	Federal Water Pollution Control Administration, a department of the United States Department of the Interior.
Rohrer	Karl R. Rohrer Associates, Inc.
T.L.C.S.	Tank Level Control System, system used on the underwater storage system to monitor tank volume and to operate the automatic control valves on the influent lines to the underwater storage tanks.
1955 I.G.L.D.	1955 International Great Lakes Datum, Elevations in feet above mean water level at Father Point, Quebec.
BOD	Five Day Biochemical oxygen demand. $(B.O.D.)_5$
COD	Chemical oxygen demand
DO	Dissolved oxygen
TS	Total solids
TSV	Total solids volatile
SS	Suspended solids
SSV	Suspended solids volatile
ml	Milliliters
mg/l	Concentration in milligrams per liter
cfs	Flow rate, cubic feet per second
mgd	Flow rate, million gallons per day
gpd	Flow rate, gallons per day
gpm	Flow rate, gallons per minute

gpcd	Gallons per capita per day
DWF	Dry weather flow
in/hr	Intensity of rainfall-inches per hour
psi	Pounds per square inch

SECTION XV

APPENDICES

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3. Water Quality Criteria 161

DISCHARGE INTO FLEXIBLE UNDERWATER TANKS

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Chairman Department of Civil Engineering
College of Engineering
University of Akron
Akron, Ohio

1967

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DISCHARGE INTO FLEXIBLE UNDERWATER TANKS

Equations given below are made on the basis of model experiments by Rohrer. If there is no tank at the end of the pipe, the formula for Q_{gpm} is:

(1) $Q = A \;\; C \;\; (2g \; h)^{1/2}$

Where: A_p is the area of the pipe,
C is .303 for the pipe used, and
h is the hydrostatic head.

If there is a flexible tank at the end of the pipe, then formula (1) is to be multiplied by a coefficient β (bag coefficient). For the different bags tested in the laboratory β is as follows:

β Rohrer Tank = .974

β Pillow Tank = .900

β appears to depend on the following:

a. Size of the bag,
b. Shape of the bag,
c. Stiffness of the bag, and
d. Relative depth of the bag under water level (expressible as depth/smaller width of tank).

The two bags tested were different in all of these conditions. Hence, the relative influence of the conditions on β can not be determined. However, testing of full scale bags under field conditions will give β sufficiently to be used in design. Direct translation of β by model scale is inadvisable now.

The experiments indicate that the rate of filling decreases as the bag fills. The following formula expresses this for:

(2) $X = \left[a - b \frac{t}{T}\right] \frac{V}{TQ}$

In this equation Q is the inflow discharge into the bag (gpm) at t, "a" and "b" are constants, t is the time of filling from start (minutes), T is the total filling time (minutes), V is the total volume of the tank under water (ft^3). X is the ratio of Q (initial) /Q (actual) at t. Experimental results indicate the following values for "a" and "b":

a =.1135 b + .028

Theoretical explanations for equation (2) would be based on concepts of underwater jet dynamics, theories of turbulence, elastic instability of membrane stresses, and the virtual mass theory of fluid dynamics.

However, the magnitude of "b" shows that the time dependent variation of the discharge is of the order $< 3\%$ of Q average - insignificant under field conditions.

Conclusions

1. The type of the bag has an influence on the rate of inflow ranging to about 10% of unhindered inflow for the pillow tank.

2. The Rohrer bag design seems to have much better hydraulic characteristics as far as the inflow in concerned, having a decreasing effect on the inflow of 2.6% only.

3. The variations of inflow due to the gradual stiffening of the filling tank is only about 3%. Under field conditions where wave actions are present this may change significantly.

PHYSICAL AND CHEMICAL PROPERTIES OF
NEOPRENE COATED NYLON
FABRIC

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In fabridam installations, the working stress allowed in the fabric is usually limited to approximately 400 pounds per linear inch, since the fabric is exposed in such installations to the atmosphere elements such as ultra-violet rays from the sun and ozone from the air which cause weathering of the fabric.

Figures 40 and 41 show test results for two-ply and one-ply neoprene rubber coated 13 ounce nylon fabric. The two-ply fabric is that used in the construction of the underwater storage tanks. The one-ply fabric is given to show the comparison in fabric strength. Possible use of this tank is anticipated if the water depth over the tank site were sufficent to allow a higher profile tank design or if internal tank pressure could be held between 0 and 0.25 psi.

Figure 42 shows the temperature ranges in which similar neoprene rubber coated fabric can be used in Fabri-tank installations. The wide range of temperatures will not be of as much importance in the underwater environment as the immersion of the fabric in water for ten and twenty year periods.

Prior to this project, neoprene rubber coated nylon fabric had not been used to any extent in contact with sewage. Little past experience could be cited for sewage applications and therefore not much is known of the chemical resistance to sewage for this particular fabric.

The physical and chemical composition of sewage varies greatly from site to site. Municipal and industrial wastes may contain certain compounds harmful to the neoprene rubber coated fabric. Tests of the liquid must be made to insure that special coatings are not necessary. However, due to the dilution of the storm water and to the residential type drainage area, possible damaging chemicals were negligible in the Sandusky, Ohio pilot facility area.

Table 14 shows those materials which have been tested by the Firestone Coated Fabrics Company to determine if the neoprene rubber coated fabric is resistant.

The rubber fabric used for the two underwater storage tanks was furnished by Firestone Coated Fabrics Company, Division of Firestone Tire and Rubber Company.

General specifications for the rubber fabric were:

> The material is to be a rubber fabric consisting of two-ply 13 ounce nylon, square woven, neoprene coated fabric with an ultimate tensile strength of 1,400 pounds per linear inch warp and fill. This material is to be supplied in 62 feet square pieces with marking lines provided for clamping locations and rolled on a mandrel to facilitate unloading and placing without damage to the fabric.

Mr. M.M. Yancy of Firestone Coated Fabrics Company supervised installation of the fabric during installation to the tank frames.

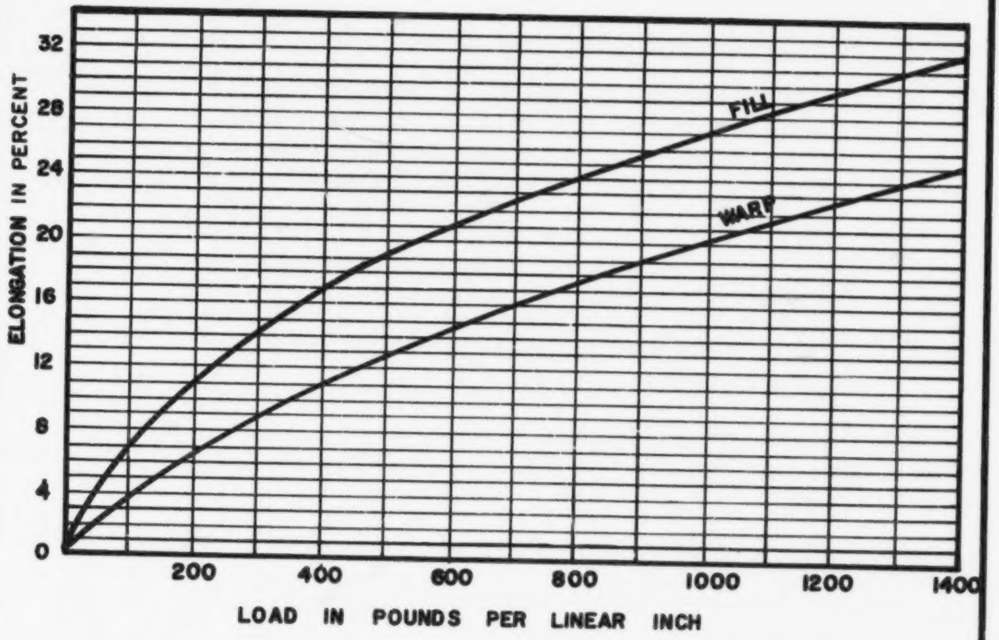
The stress bearing component of the neoprene rubber coated fabric is the nylon fabric. Neoprene base compounds have been chosen because their weathering characteristics are excellent and the life expectancy of the fabric has been determined to be in the neighborhood of twenty years in Imbertson Fabridam installations.

Fabridam material similar to that used in the underwater storage tanks has the following physical properties:

Construction	Wt. Per Sq. Yd.	Puncture Resistance in #	Ultimate Breaking Strength #/lin.in.
2-ply hvy. duty	8.3 lbs.	370	1400

The test used in measuring puncture resistance is per spec. MIL-T-6396 B.

The abrasive resistance of fabridam construction has been excellent. During a six month test of vibrating a rubber coated fabric sample in a box with a mixture of sand, gravel, and water, no appreciable wear was detected. The rubber coatings have been accepted as standard protective liners and covers for internal parts of sand blasting, grit blasting, and similar abrasive inducing equipment.

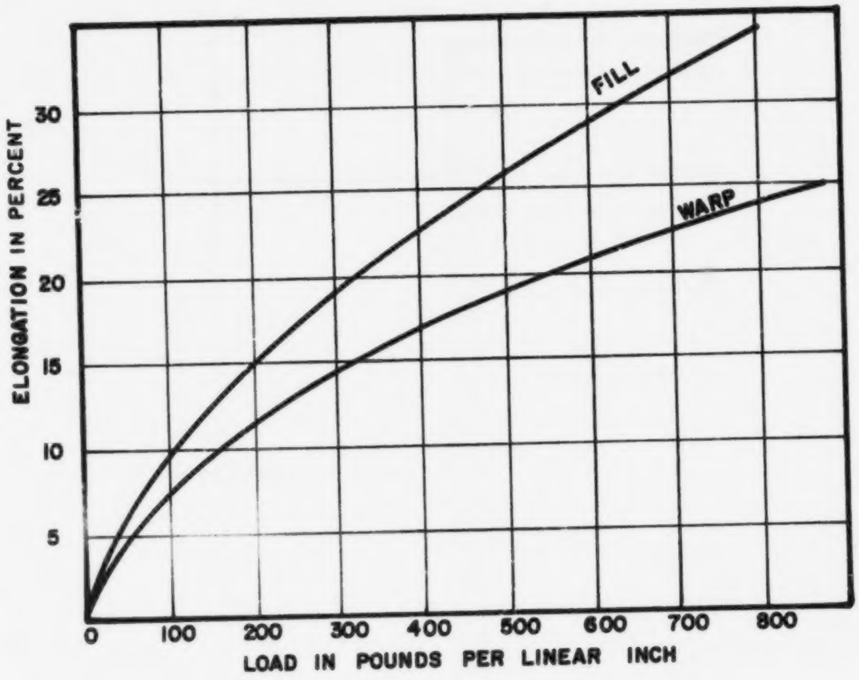


TWO - PLY NEOPRENE COATED
13 OZ. NYLON FABRIC

FIRESTONE COATED FABRICS COMPANY

KARL R. ROHRER ASSOCIATES, INC.
AKRON, OHIO

FIGURE 40

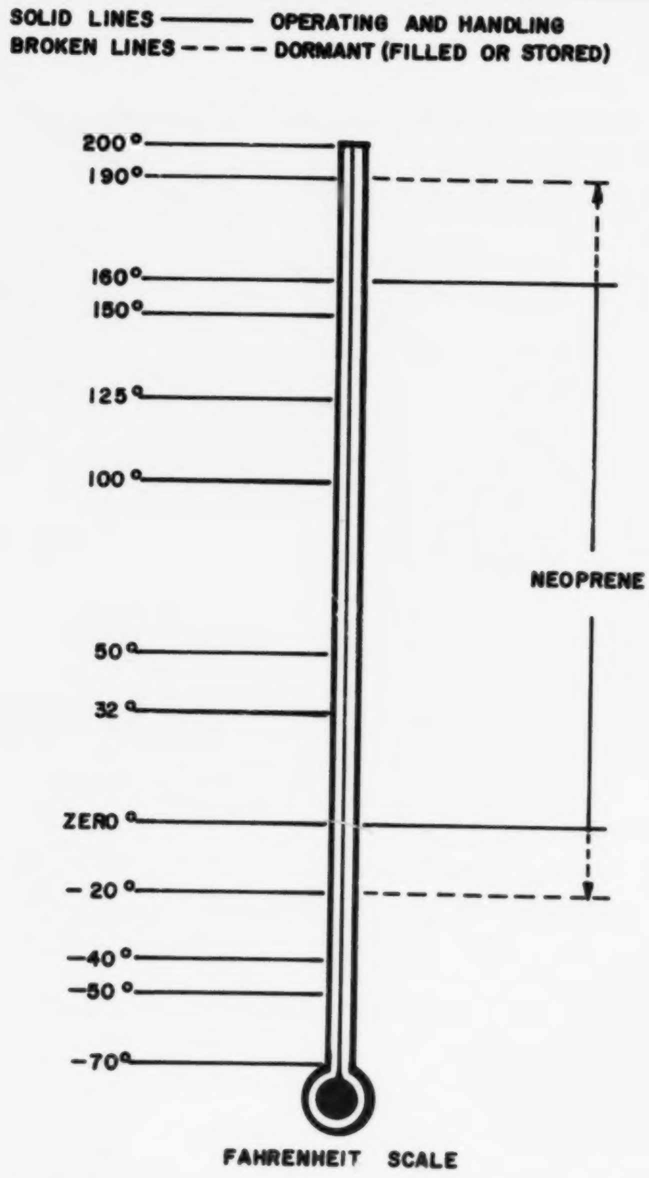


SINGLE-PLY NEOPRENE COATED
13 OZ. NYLON FABRIC

FIRESTONE COATED FABRICS COMPANY

KARL R. ROHRER ASSOCIATES, INC.
AKRON, OHIO

FIGURE 41


SOLID LINES ——— OPERATING AND HANDLING
BROKEN LINES - - - - DORMANT (FILLED OR STORED)
200°
190°
160°
150°
125°
100°
50°
32°
ZERO°
- 20°
-40°
-50°
-70°
NEOPRENE
FAHRENHEIT SCALE

FABRITANK USE TEMPERATURE RANGE
FIRESTONE COATED FABRICS COMPANY

KARL R. ROHRER ASSOCIATES, INC.
AKRON, OHIO

FIGURE 42

TABLE 14

FABRITANK POLYMERS

Firestone Coated Fabrics Company

Revised 11/10/64

Type	Material	Chemical Group	Trade or Popular Name	General Use
NP	Neoprene	Chloroprene	GR-M Neoprene	Process Water, Near Neutral Fertilizer Solutions, Moderate Chemicals

FABRITANK CONTAINERS

Chemical Resistance

Revised 7/24/64

R - Resists - Tank is suitable for containing the indicated material.

NR - Not Resistant - Tank is not suitable for containing the indicated fluid.

Notes: (a) Initial discoloration of fluid may take place.

(b) Taste of contained fluid may be affected.

Table 14 Continued

Material	Concentration	NP
Acid, Acetic	Up to 80%	NR
Acid, Boric	---	R
Acid, Chromic	Any	NR
Acid, Citric (not potable)	Any	R
Acid Hydorchloric	Any	NR
Acid, Lactic (not potable)	Any	R
Acid, Oleic	---	NR
Acid, Phosphoric, Cold	Up to 60%	NR
Acid, Sulfuric	Any	NR
Acid, Tartaric (not comestible)		R
Acid, Tannic - to 10%	Any	R
Acetone		NR
Alums of Ammonia		R
Alums of Chromium		NR
Alums of Pottassium		R
Ammonium Chloride		R
Ammonium Phosphate, Mono, Di, Tri		R
Ammonium Sulfate		R
Amyle Alcohol		R
Aniline		NR
ASTM Oils 1, 2, & 3		NR
Asphalt Emulsions		R
Asphalt (160 to 180°F)		NR
Barium Chloride		R
Barium Hydroxide		R
Barium Sulfate		R
Beet Sugar Liquors		R (b)
Benzene		NR
Borax		R
Butadience Styrene (SBR) Latex		R
Bunker C Fuel Oil (160° - 180° F)		NR
Calcium Acetate		R
Calcium Bisulfite		R
Calcium Chloride		R
Calcium Hydroxide		R
Calcium Nitrate		R
Cane Sugar Liquors - (Not Comestible)		R
Carbon Tetrachloride		NR
Castor Oil - (Not Comestible)		NR
Chlorinated Solvents		NR
Chrome Plating Solutions		NR
Cocoanut Oil - (Not Comestible)		R (a)
Copper Sulfate		R
Creosote, Coal Tar		NR
Crude Oil, Sour		NR
Denatured Alcohol (Methanol)		R (a)

Table 14 Continued

Material	Concentration	NP
Diesel Fuel		NR
Decalin		NR
Dibutrl Phthalate		NR
Diethylene Glycol		R (a)
Dimethyl Hydrazine		NR
Dioctrl Phthalate		NR
Dipentene		NR
Dowtherm		NR
Ester. Plasticizers, Monomeric		NR
Ethyl Acetate		NR
Ethyl Alcohol (Ethanol)		R (a)
Ethylene Glycol		R (a)
Fatty Acids		NR
Formalin	to 37%	R
Gasoline, Automotive		NR
Gasoline, Aircraft		NR
Gelatin (Not Comestible		R
Glucose (Not Comestible	Any	R
Glycerol		R (a)
n-Hexane		NR
Hydrazine	Any	NR
Halowax Oil		NR
Kerosene		R
Ketones as a Class		NR
Latex - FRS		R
Latex - Hevea		R
Linseed Oil		NR
Methyl Alcohol		R (a)
Mesityl Oxide		NR
Mineral Spirits		NR
Motor Oil		NR
Naptha, VM & P		NR
Nickel Chloride		R
Nickel Sulfate		R
Nitro Parafins		NR
Olive Oil (Not Comestible)		NR
Peanut Oil (Not Comestible)		NR
Phenol, Liquid	Any	NR
Pine Oil, if flash point exceeds	80° F	NR
Pine Oil		NR
Potassium Acetate	Any	R
Potassium Chloride	Any	R
Potassium Sulfate	Any	R
Propyl Alcohol		R (a)
Salt Water		R
Sewage		

Table 14 Continued

Material	Concentration	NP
Shell Sol 140		NR
Skelly Solve, B,C,E,		NR
Skydrol 500		NR
Sodium Chloride	Any	R
Sodium Sulfate	Any	R
Styrene		NR
Sucrose Solutions (Not Comestible)		R
Sulfur		R
Tetralin		NR
Toulene		NR
Transformer Oil to 160°F		NR
Turpentine		NR
Tricreyl Phosphate		NR
Urea - Formaldehyde Resins, Liquid		R (a)
Water - Brine Process, Beverage (Not Potable)		R
Water, Process		R
Water, Potable		R (b)
Xylene		NR

WATER POLLUTION CONTROL BOARD

DEPARTMENT OF HEALTH

COLUMBUS, OHIO

Water Quality Criteria Adopted by the Board April 11, 1967

For Lake Erie and The Interstate Waters Thereof

The Ohio Water Pollution Control Board hereby adopts the following water quality criteria for Lake Erie and the interstate waters thereof which may affect the State of Michigan, the Commonwealth of Pennsylvania, the State of New York, and the Province of Ontario of the Dominion of Canada.

Water Quality - Conditions and Criteria

All Waters. All the waters considered herein shall meet the following conditions at all times:

1. They shall be free from substances attributable to municipal, industrial, or other discharges that will settle to form putrescent or otherwise objectionable sludge deposits;

2. They shall be free from floating debris, oil, scum, and other floating materials attributable to municipal, industrial or other discharges in amounts sufficient to be unsightly or deleterious;

3. They shall be free from materials attributable to municipal, industrial, or other discharges producing color, odor, or other conditions in such degree as to create a nuisance; and,

4. They shall be free from substances attributable to municipal, industrial, or other discharges in concentrations or combinations which are toxic or harmful to human, animal, plant, or aquatic life.

Lake Erie Water Quality Criteria for Various Uses are: (1) the Stream-Water Quality Criteria for Various Uses adopted by the Ohio Water Pollution Control Board on June 14, 1966 copy attached, which shall apply as a minimum to all Lake Erie waters in Ohio, and (2) the existing lake water quality which shall apply, where better than, the criteria for streams adopted by the Board. The existing lake water quality shall be as reported by the Federal Water Pollution Control Administration in the chapter on Water Quality in the report "Program for Water Pollution Control - Lake Erie-1967."

Lake Erie outside the established harbors at Lorain, Cleveland and Ashtabula shall meet the Lake Erie Quality Criteria for all uses.

The Lorain, Cleveland, and Ashtabula harbor waters in Lake Erie shall meet the Lake Erie Quality Criteria for industrial water supply and aquatic life. (A).

Implementation and Enforcement Plan

The Ohio Water Pollution Control Board, under the provisions of Sections 6111.01 to 6111.08, 6111.31 to 6111.38, and 6111.99. Ohio Revised Code, has authority to control, prevent, and abate pollution in the waters of this state. In accordance with such authority, the Board hereby adopts the following program and requirements for the prevention, control, and abatement of new or existing pollution of the waters of Lake Erie.

1. The "Recommendations and Conclusions - August 12, 1965" agreed upon by the conferees from Michican, Indiana, Ohio, Pennsylvania, New York, and the U.S. Public Health Service following a conference under Section 8 of the Federal Water Pollution Control Act in the matter of pollution of the interstate and Ohio intrastate waters of Lake Erie and its tributaries held in Cleveland, Ohio, August 3-6, 1965, and in Buffalo, New York, August 10-12, 1965, and "Report of the Lake Erie Enforcement Conference Technical Committees -March, 1967" are included as a part of this program insofar as applicable to Lake Erie waters in Ohio;

2. All plans and proposals for abatement or correction of pollution will be approved by the Ohio Department of Health as required by law and such approvals shall constitute approval by the Board;

3. All sewage will be given secondary treatment (biochemical oxidation), and the facilities to provide such treatment

will be constructed and placed in operation without delay, and in no instance later than the dates specified in the attached lists;

4. All effluents will be satisfactorily disinfected to meet the criteria for Lake Erie water uses and the facilities to provide such disinfection will be installed without delay;

5. All industrial wastes will be adequately treated to meet the Lake Erie water quality conditions and criteria and the facilities to provide such treatment will be constructed and placed in operation without delay, and in no instance later than the dates specified in the attached lists;

6. Local programs will be initiated to control and reduce pollution resulting from (a) bypassing, (b) spillages, and (c) discharges resulting from construction or breakdowns;

7. Necessary studies will be made and, where feasible, plans and construction programs will be developed as rapidly as possible for reducing pollution from combined sewer overflows;

8. Where necessary to improve water quality and to reduce algal growths in Lake Erie, supplementary treatments of waste-waters will be provided to the fullest extent consistent with research and technological advances;

9. Where necessary to protect recreational areas of Lake Erie, studies will be made by the responsible agencies, and plans and construction programs will be developed as rapidly as possible for improvements such as (a) elimination, treatment or diversion of combined and storm sewer discharges from beaches and other recreational areas, (b) diversion of all effluent discharges, both sewage and industrial wastes, from areas where they may adversely affect recreational waters, and (c) elimination of the physical entrapment of storm water, marsh drainage, debris, and other pollutants at beach areas;

10. The Lake Erie water quality monitoring program will be expanded as outlined in the attached report to adequately provide assurances of compliance with these criteria.

Furthermore, the Board and the Ohio Department of Health will:

1. Encourage and assist other agencies such as the Ohio Water Commission and the Soil Conservation Service, U.S. Department of Agriculture, in the implementation of effective soil erosion control programs, and programs for the reduction of the run-off of phosphorous, nitrogen compounds, and pesticides;

2. Encourage the enactment of State legislation prohibiting the discharge of untreated wastewater from pleasure craft to the Lake Erie waters in Ohio, and requiring adequate waste disposal facilities at marina along Lake Erie; and,

3. Seek adequate legislation prohibiting the open dumping of garbage, trash, and other deleterious refuse along the shores of Lake Erie.

Enforcement of these requirements will be carried out by means of the respective permits issued to municipalities, counties, industries, and other entities discharging to the Lake Erie waters of Ohio considered herein, and failure to comply with the permit conditions will result in legal action in accordance with the provisions of laws.

WATER POLLUTION CONTROL BOARD

OHIO DEPARTMENT OF HEALTH

COLUMBUS, OHIO

Resolution Regarding Amended Criteria of Stream-Water Quality For Various Uses Adopted by the Board on October 1, 1967.

WHEREAS, Section 6111.03, of the Ohio Revised Code, provides, in part, as follows:

> "The water pollution control board shall have power:
>
> (A) To develop programs for the prevention, control and abatement of new or existing pollution of the waters of the state; . . ." and

WHEREAS, Primary indicators of stream-water quality are needed as guides for appraising the suitability of surface waters in Ohio for various uses; and

WHEREAS, The stream-water quality criteria for various uses and minimum conditions applicable to all waters adopted by the Board on June 14, 1966, have been amended by the Ohio River Valley Water Sanitation Commission;

THEREFORE BE IT RESOLVED, That the following amended stream-water quality criteria for various uses, and minimum conditions applicable to all waters, are hereby adopted in accordance with amendments of the Ohio River Water Sanitation Commission.

Minimum Conditions Applicable to All Waters at All Places and At All Times

1. Free from substances attributable to municipal, industrial or other discharges, or agricultural practices that will settle to form putrescent or otherwise objectionable sludge deposits.

2. Free from floating debris, oil, scum and other floating materials attributable to municipal, industrial, or other discharges, or agricultural practices in amounts sufficient to be unsightly or deleterious.

3. Free from materials attributable to municipal, industrial or other discharges, or agricultural practices producing color, odor or other conditions in such degree as to create a nuisance.

4. Free from substances attributable to municipal, industrial or other discharges, or agricultural practices in concentrations or combinations which are toxic or harmful to human, animal, plant or aquatic life.

STREAM-QUALITY CRITERIA

For Public Water Supply

The following criteria are for evaluation of stream quality at the point at which water is withdrawn for treatment and distribution as a potable supply:

1. Bacteria: Coliform group not to exceed 5,000 per 100 ml as a monthly average value (either MPN or MF count); nor exceed this number in more than 20 percent of the samples examined during any month; nor exceed 20,000 per 100 ml in more than five percent of such samples.

2. Threshold-odor Number: Not to exceed 24 (at 60 deg.C) as a daily average.

3. Dissolved solids: Not to exceed 500 mg/l as a monthly average value, nor exceed 750 mg/l at any time.

4. Radioactivity: Gross beta activity not to exceed 1,000 picocuries per liter (pCi/l), nor shall activity from dissolved strontium-90 exceed 10 pCi/l, nor shall activity from dissolved alpha emitters exceed 3 pCi/l.

5. Chemical constituents: Not to exceed the following specified concentrations at any time:

Constituent	Concentration (mg/l)
Arsenic	0.05
Barium	1.0
Cadmium	0.01
Chromium (hexavalent)	0.05
Cyanide	0.025
Fluoride	1.0
Lead	0.05
Selenium	0.01
Silver	0.05

For Industrial Water Supply

The following criteria are applicable to stream water at the point at which the water is withdrawn for use (either) with or without treatment) for industrial cooling and processing:

1. Dissolved Oxygen: Not less than 2.0 mg/l as a daily average value, nor less than 1.0 mg/l at any time.
2. pH: Not less than 5.0 nor greater than 9.0 at any time.
3. Temperature: Not to exceed 95 deg. F at any time.
4. Dissolved solids: Not to exceed 750 mg/l as a monthly average value, nor exceed 1,000 mg/l at any time.

For Aquatic Life A

The following criteria are for evaluation of conditions for the maintenance of a well-balanced, warm-water fish population. They are applicable at any point in the stream except for areas immediately adjacent to outfalls. In such areas cognizance will be given to opportunities for the admixture of waste effluents with stream water:

1. Dissolved Oxygen: Not less than 5.0 mg/l during at least 16 hours of any 24-hour period, nor less than 3.0 mg/l at any time.
2. pH: No values below 5.0 nor above 9.0 and daily average (or median) values preferabley between 6.5 and 8.5.
3. Temperature: Not to exceed 93 deg. F at any time during the months of May through November, and not to exceed 73 deg. F at any time during the months of December through April.
4. Toxic Substances: Not to exceed one-tenth of the 48-hour median tolerance limit, except that other limiting concentrations may be used in specific cases when justified on the basis of available evidence and approved by the appropriate regulatory agency.

For Aquatic Life B

The following criteria are for evaluation of conditions for the maintenance of desirable biological growths and, in limited stretches of a stream, for permitting the passage of fish through the water, except for areas immediately adjacent to outfalls. In such areas cognizance will be given to opportunities for admixture of effluents with stream water.

1. Dissolved Oxygen: Not less than 2.0 mg/l as a daily average value, nor less than 1.0 mg/l at any time.

2. pH: Not less than 5.0 nor greater than 9.0 at any time.

3. Temperature: Not to exceed 95 deg. F at any time.

4. Toxic Substances: Not to exceed one-tenth of the 48-hour median tolerance limit, except that other limiting concentrations may be used in specific cases when justified on the basis of available evidence and approved by the appropriate regulatory agency.

For Recreation

The following criterion is for evaluation of conditions at any point in waters designated to be used for recreational purposes, including such water-contact activities as swimming and water skiing:

> Bacteria: Coliform group not to exceed 1,000 per 100 ml as a monthly average value (either MPN or MF count); nor exceed this number in more than 20 percent of the samples examined during any month; nor exceed 2,400 per 100 ml (MPN or MF count) on any day.

For Agricultural Use and Stock Watering

The following criteria are applicable for the evaluation of stream quality at places where water is withdrawn for agricultural use or stock-watering purposes:

1. Free from substances attributable to municipal, industrial or other discharges, or agricultural practices that will settle from putrescent or otherwise objectionable sludge deposits.

2. Free from floating debris, oil, scum, and other floating materials attributable to municipal, industrial or other discharges, or agricultural practices in amounts sufficient to be unsightly or deleterious.

3. Free from materials attributable to municipal, industrial or other discharges, or agricultural practices producing color, odor or other conditions in such degree as to create a nuisance.

4. Free from substances attributable to municipal, industrial or other discharges or agricultural practices in concentrations or combinations which are toxic or harmful to human, animal, plant or aquatic life.

1 *Accession Number*

2 *Subject Field & Group*
05D

SELECTED WATER RESOURCES ABSTRACTS
INPUT TRANSACTION FORM

5 *Organization*
Karl R. Rohrer Associates, Inc.
529 Grant Street
Akron, Ohio 44311

6 *Title*
Underwater Storage of Combined Sewer Overflows

10 *Author(s)*
Rohrer, Karl R.
Bandy, William J.,Jr.

16 *Project Designation*
EPA Project No. 11022 ECV.
Contract No. 14-12-143

21 *Note*

22 *Citation*
Water Pollution Control Research Series - 11022 ECV 9/71

23 *Descriptors (Starred First)*
Water Pollution Control
Combined Sewers
Underwater Storage
Storm Overflow

25 *Identifiers (Starred First)*
Flexible Tank

27 *Abstract*

The purpose of this study was to demonstrate off-shore underwater temporary storage of storm overflow from a combined sewer in flexible tanks. Site selection, model testing, system design, construction, and one year's operation were conducted under the study.

A pilot demonstration facility was constructed in Sandusky, Ohio where combined sewer overflow from a 14.86-acre residential drainage area was directed to two-100,000 gallon collapsible tanks anchored underwater in Lake Erie. The stored overflows were pumped back to the sewer system after a storm event for subsequent treatment. During the year's operation, a total of 988,000 gallons of storm overflow was contained and returned for treatment.

As constructed, the facility cost was about $1.88 per gallon of storage capacity while future projections indicate costs of less than $0.40 per gallon possible.

Evaluation of the underwater storage system in controlling combined sewer pollution, comparison of cost with other storage methods and other combined sewer pollution control methods, operational difficulties and recommendations of an improved system are included in the study report.

This report was submitted in fulfillment of Contracts 14-12-25 and 14-12-143 between Water Quality Research, Environmental Protection Agency, and Karl R. Rohrer Associates, Inc.

Abstractor William J. Bandy,Jr.
Institution Karl R. Rohrer Associates, Inc.

WR 102 (REV JULY 1969)
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Continued from inside front cover....

11022	—	08/67	Phase I - Feasibility of a Periodic Flushing System for Combined Sewer Cleaning
11023	—	09/67	Demonstrate Feasibility of the Use of Ultrasonic Filtration in Treating the Overflows from Combined and/or Storm Sewers
11020	—	12/67	Problems of Combined Sewer Facilities and Overflows, 1967 (WP-20-11)
11023	—	05/68	Feasibility of a Stabilization-Retention Basin in Lake Erie at Cleveland, Ohio
11031	—	08/68	The Beneficial Use of Storm Water
11030	DNS	01/69	Water Pollution Aspects of Urban Runoff, (WP-20-15)
11020	DIH	06/69	Improved Sealants for Infiltration Control, (WP-20-18)
11020	DES	06/69	Selected Urban Storm Water Runoff Abstracts, (WP-20-21)
11020	—	06/69	Sewer Infiltration Reduction by Zone Pumping, (DAST-9)
11020	EXV	07/69	Strainer/Filter Treatment of Combined Sewer Overflows, (WP-20-16)
11020	DIG	08/69	Polymers for Sewer Flow Control, (WP-20-22)
11023	DPI	08/69	Rapid-Flow Filter for Sewer Overflows
11020	DGZ	10/69	Design of a Combined Sewer Fluidic Regulator, (DAST-13)
11020	EKO	10/69	Combined Sewer Separation Using Pressure Sewers, (ORD-4)
11020	—	10/69	Crazed Resin Filtration of Combined Sewer Overflows, (DAST-4)
11024	FKN	11/69	Stream Pollution and Abatement from Combined Sewer Overflows - Bucyrus, Ohio, (DAST-32)
11020	DWF	12/69	Control of Pollution by Underwater Storage
11000	—	01/70	Storm and Combined Sewer Demonstration Projects - January 1970
11020	FKI	01/70	Dissolved Air Flotation Treatment of Combined Sewer Overflows, (WP-20-17)
11024	DOK	02/70	Proposed Combined Sewer Control by Electrode Potential
11023	FDD	03/70	Rotary Vibratory Fine Screening of Combined Sewer Overflows, (DAST-5)
11024	DMS	05/70	Engineering Investigation of Sewer Overflow Problem - Roanoke, Virginia
11023	EVO	06/70	Microstraining and Disinfection of Combined Sewer Overflows
11024	—	06/70	Combined Sewer Overflow Abatement Technology